Load Rating Assessment of a 56-Year-Old Short-Span Composite Steel Girder Bridge in a Coastal Environment: Brai 2 Bridge

Jampari Manukoa[1]*

[1] Roads and Bridges Study Program, Akademi Teknik Kupang, Kupang, Indonesia

* Corresponding author: jampari_manukoa@gmail.com

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ABSTRACT

Brai 2 Bridge is a 5.00 m span composite steel girder bridge built in 1970 and located approximately 30 m from the shoreline, where its steel structure is exposed to a highly corrosive marine atmosphere. By the 2026 evaluation, the bridge had been in service for 56 years, exceeding the 50-year design service life commonly adopted for similar bridges in Indonesia, yet no formal load rating had been performed. This study evaluates the structural capacity of the bridge superstructure while accounting for long-term corrosion-induced section loss. Atmospheric corrosion was predicted using the Albrecht and Naeemi power-law model, and Rating Factors (RFs) were calculated in accordance with SNI 1725:2016 and Indonesian Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016. The assessment was independently verified using manual Lever Rule calculations, a 2D grillage model, and SAP2000 finite-element modeling. The predicted steel thickness loss was approximately 3.40 mm (17%), reducing nominal composite flexural capacity from about 1130 kN.m to 973.5 kN.m. For this short span, the two-lane "T" Truck load governed over the "D" Load. The overall governing RF was 1.64 for support shear using the official procedure, while flexural RFs ranged from 1.77 using the Lever Rule to 2.90 using SAP2000. Because all RFs were at least 1.0, the superstructure remains structurally adequate without load restriction, although its safety margin has decreased. The study demonstrates the value of multi-method verification for aging short-span bridges in corrosive coastal environments.

Keywords: *load rating; composite bridge; atmospheric corrosion; short span; rating factor*

ABSTRAK

Jembatan Brai 2 merupakan jembatan gelagar baja komposit dengan bentang 5,00 m yang dibangun pada tahun 1970 dan terletak sekitar 30 m dari garis pantai, sehingga struktur bajanya terpapar atmosfer laut yang sangat korosif. Pada evaluasi tahun 2026, jembatan telah beroperasi selama 56 tahun, melampaui umur layan rencana 50 tahun yang umumnya digunakan untuk jembatan sejenis di Indonesia, namun belum pernah dilakukan penilaian kapasitas beban secara formal. Penelitian ini mengevaluasi kapasitas struktur atas jembatan dengan memperhitungkan kehilangan penampang akibat korosi jangka panjang. Korosi atmosfer diprediksi menggunakan model power-law Albrecht dan Naeemi, sedangkan Rating Factor (RF) dihitung sesuai SNI 1725:2016 dan Surat Edaran Menteri PUPR No. 03/SE/M/2016. Penilaian diverifikasi secara independen menggunakan perhitungan manual Lever Rule, model grillage 2D, dan pemodelan elemen hingga SAP2000. Kehilangan tebal baja diprediksi sekitar 3,40 mm (17%), yang menurunkan kapasitas lentur nominal komposit dari sekitar 1130 kN.m menjadi 973,5 kN.m.

Untuk bentang pendek ini, beban Truk “T” dua lajur lebih menentukan dibandingkan Beban “D”. RF yang mengontrol adalah 1,64 pada geser tumpuan menggunakan prosedur resmi, sedangkan RF lentur berkisar dari 1,77 dengan Lever Rule hingga 2,90 dengan SAP2000. Karena seluruh nilai RF sekurang-kurangnya 1,0, struktur atas masih memadai secara struktural tanpa pembatasan beban, meskipun margin keamanannya telah berkurang. Studi ini menunjukkan pentingnya verifikasi multi-metode untuk jembatan bentang pendek yang menua di lingkungan pesisir yang korosif.

Kata Kunci: *penilaian beban; jembatan komposit; korosi atmosfer; bentang pendek; faktor penilaian*

1. INTRODUCTION

Bridge infrastructure is a vital component of road networks that supports the economic and social activities of communities. A substantial proportion of existing bridges in Indonesia, particularly on regency and rural road networks, consists of short-span bridges (< 20 m) constructed during the 1960s-1980s. Many of these structures are now approaching or have exceeded their intended service lives. This condition creates an urgent need for systematic structural-capacity evaluation to ensure that these bridges can safely accommodate current traffic loading requirements.

Brai 2 Bridge represents this condition. It is a composite steel girder bridge with a reinforced-concrete deck, constructed in 1970 and still in service in the 2026 evaluation year. The bridge has therefore been operating for 56 years, exceeding the 50-year design service life commonly adopted for similar bridges in Indonesia. It is located in a coastal area approximately 30 m from the shoreline, exposing its steel girders to a highly corrosive marine atmospheric environment because of the high chloride (salt) content in the air. Visual observations indicate substantial corrosion of the steel girders, although no deterioration has yet been observed that clearly impairs the structural function of the bridge.

Evaluating an existing bridge that has exceeded its design service life and is located in a corrosive environment requires a different approach from the design of a new bridge because the actual structural capacity has deteriorated as a result of aging and environmental exposure. In accordance with Indonesian Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016 on Guidelines for Determining Bridge Load Rating for Existing Bridges, the capacity of an existing bridge should be evaluated periodically by comparing its current structural capacity, corrected for deterioration such as corrosion, with the traffic loads prescribed by the current standard, SNI 1725:2016, rather than with the loading standard used when the bridge was originally designed.

Bridge Rating Factor (RF) studies have generally focused on medium- and long-span bridges. Short-span bridges (≤ 5-10 m), although they represent a large proportion of the national bridge population, have received comparatively limited in-depth attention, particularly in studies that combine long-term corrosion effects with multi-method verification using manual calculations, grillage modeling, and finite-element analysis. For very short spans, several standard load-distribution approaches, including stiffness-based AASHTO LRFD girder-distribution formulas, fall outside their stated range of applicability. Alternative procedures such as the Lever Rule are therefore required and should be independently verified.

Accordingly, this study aims to: (1) predict the loss of steel-girder section thickness caused by atmospheric corrosion in a coastal environment; (2) calculate the corrected nominal flexural and shear capacities of the existing composite section; (3) determine the bridge Rating Factor (RF) using both a practical general equation and the official procedure in Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016; (4) verify the load-distribution results using the Lever Rule, a 2D grillage model, and SAP2000 finite-element modeling; and (5) determine the operational feasibility of the bridge and recommend follow-up actions. The scope is limited to the superstructure, namely the composite girders and deck slab. The substructure, including abutments, piers, foundations, and bearings, is outside the scope and requires a separate assessment.

The novelty of this study lies in the simultaneous consideration of three characteristics that are rarely examined together: a very short span that falls outside the applicability range of standard load-distribution formulas, a structural age exceeding the intended design service life, and severe exposure to a highly corrosive coastal environment. These conditions are assessed through three independent analytical approaches. The results are expected to provide methodological insight for short-span bridge asset management in Indonesia and practical guidance for engineering service providers and road-management agencies in prioritizing rehabilitation of aging existing bridges.

2. METHODS

2.1 Research Design

This research adopts a quantitative case-study approach to evaluate the structural capacity of an existing bridge. The study stages comprise collection of bridge geometry and field-condition data, prediction of corrosion-induced thickness loss, calculation of the corrosion-corrected composite-section capacity, traffic-load analysis in accordance with SNI 1725:2016, calculation of the Rating Factor (RF) in accordance with Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016, and verification using a 2D grillage model and SAP2000 finite-element modeling.

2.2 Study Area / Object of Study

The object of study is Brai 2 Bridge, a composite steel girder bridge with a reinforced-concrete deck and a simply supported single span of L = 5.00 m. The carriageway width is 6.00 m and the total out-to-out width is 7.20 m. The superstructure consists of six WF steel girders with variable transverse spacing of 0.70 m (edge) - 1.10 m - 1.10 m - 1.40 m (center) - 1.10 m - 1.10 m - 0.70 m (edge). The bridge is located approximately 30 m from the shoreline and is therefore exposed to a highly corrosive marine atmosphere, corresponding to approximately C5/CX corrosivity under ISO 9223:2012. General bridge data are summarized in Table 1.

Table 1. General data for Brai 2 Bridge

Parameter	Data
Superstructure type	Composite steel girders with concrete deck slab
Span	5.00 m (simple span)
Carriageway / total width	6.00 m / 7.20 m
Number of main girders	6
Year constructed / evaluated	1970 / 2026
Structural age at evaluation	56 years
Location	Coastal area, approximately 30 m from the shoreline
Steel-girder condition	Substantial corrosion; no observed local failure

Source: Author survey data and analysis, 2026.

The field survey was conducted on Monday, 13 July 2026, at coordinates 8°48′48.342″S, 121°35′49.644″E, on the Aegela-Ende City Boundary road section. The survey documented the existing bridge condition from both the roadway and underside of the structure, as shown in Figure 1. Figure 1(b) shows that the composite steel girders had been coated with protective paint in May 2026, approximately two months before the survey, to slow further corrosion. However, this recent coating constitutes a preventive maintenance action and does not restore the cumulative section-thickness loss that occurred during the preceding 56 years of service. Therefore, the predicted corrosion-induced thickness loss presented in Section 3.1 remains relevant as the basis for evaluating structural capacity.

Figure 1. Field survey documentation of Brai 2 Bridge, 13 July 2026: (a) bridge and safety barrier viewed from the roadway; (b) composite steel girders viewed from beneath the structure, coated with protective paint in May 2026 as a preventive measure to reduce further corrosion.

Source: Author field-survey documentation, 13 July 2026.

The WF structural steel has a yield strength of fy = 240 MPa, and the deck concrete has a compressive strength of fc' = 20 MPa. The initial WF steel-section dimensions for each girder are: overall depth ds = 40 cm, flange width bf = 14 cm, flange thickness tf = 2 cm, web thickness tw = 2 cm, clear web depth d = 36 cm, and initial steel area As = 128 cm2. The concrete deck slab is 25 cm thick, with an effective tributary width bc = 110 cm for an interior girder. The composite-section sketch and bridge cross-section are shown in Figures 2 and 3.

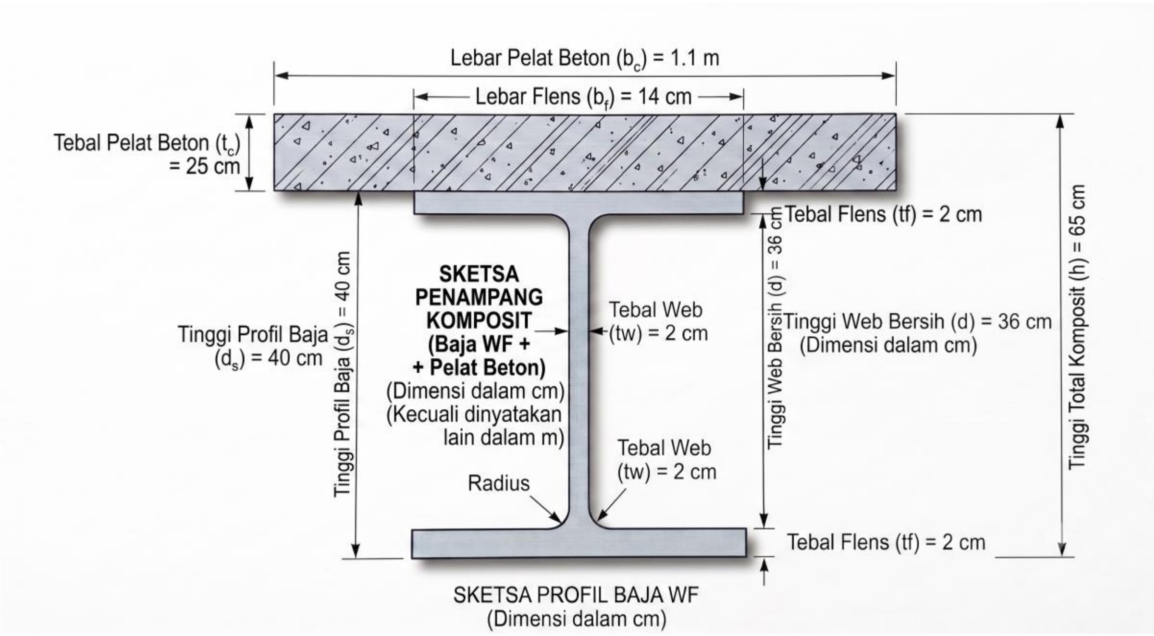


Figure 2. Composite-section sketch (WF steel section and concrete slab) of Brai 2 Bridge; dimensions in cm.

Source: Author analysis and technical drawing, 2026.

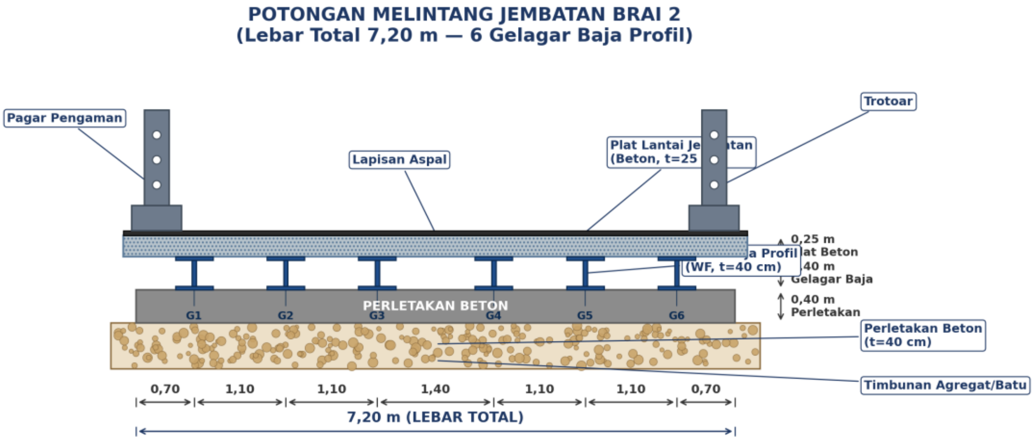


Figure 3. Cross-section of Brai 2 Bridge: 6.00 m carriageway, 7.20 m total width, and six WF steel girders. Source: Author analysis and technical drawing, 2026.

2.3 Data Collection

Primary data were obtained from visual field observations conducted in the 2026 evaluation year, including girder corrosion, asphalt-surface condition, and the general structural condition. Secondary data included bridge geometry and material specifications obtained from technical documentation provided by the road-management agency, together with the loading provisions and bridge-rating procedures specified in the applicable national standards, namely SNI 1725:2016 and Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016.

2.4 Data Analysis

Data analysis was performed in several stages. First, atmospheric corrosion-induced thickness loss was predicted using the power-law model proposed by Albrecht and Naeemi (1984):

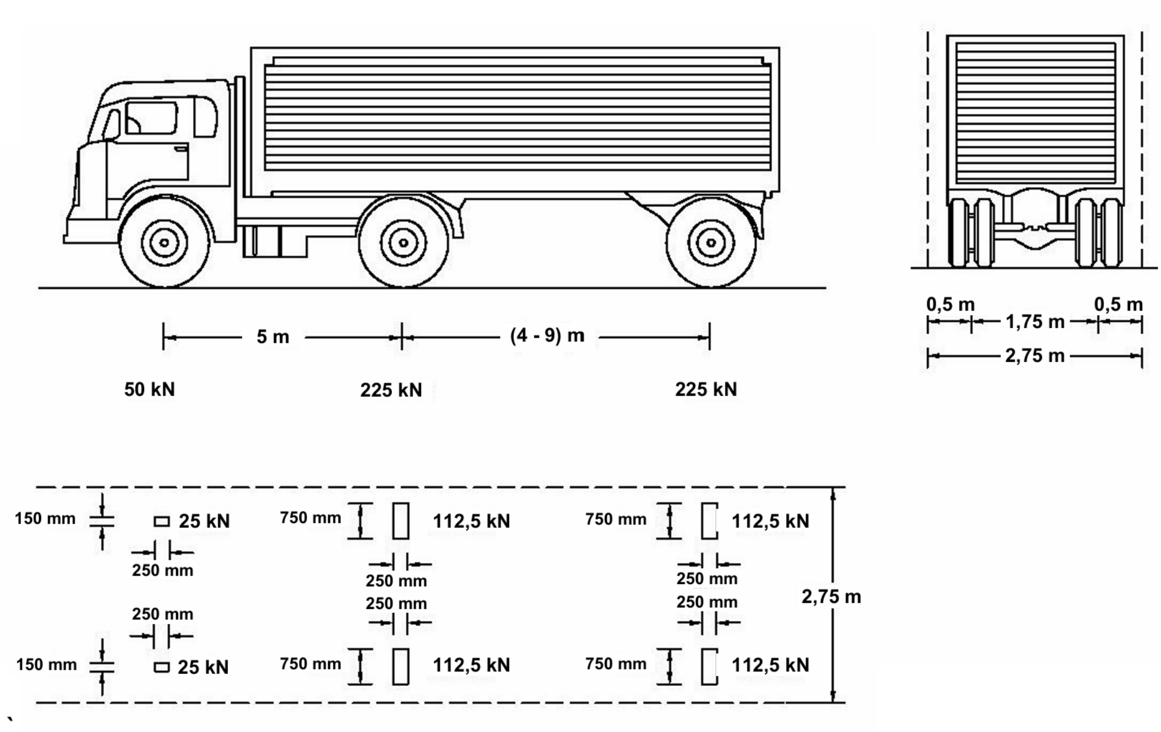
$$C(t) = A \times tB$$

where C is the thickness loss per exposed surface (micrometers), t is the exposure duration (years), and A and B are environmental coefficients. For a marine environment, A = 70.6 and B = 0.79 were used.

Second, the corrected nominal flexural capacity of the composite section was calculated using plastic analysis with an equivalent rectangular stress block and assuming full composite action between the steel section and the concrete slab. The procedure included calculation of the tensile force in the fully yielded steel section, determination of the plastic neutral axis (PNA), calculation of the concrete compression-block depth, determination of the internal moment arm, and calculation of the corrected nominal moment capacity (Mn'). The nominal web shear capacity was calculated using the corrosion-corrected web thickness, including a web-slenderness check and the shear-buckling reduction factor Cv, conservatively assuming an unstiffened web.

Third, traffic-load analysis included dead load from the self-weight of the corroded steel section, concrete slab, and a 7 cm asphalt layer representing the actual field condition, together with live loads consisting of the "D" Load, comprising a uniformly distributed load (BTR) and a line load (KEL), and the 500 kN "T" Truck load applied in two lanes in accordance with SNI 1725:2016. The Dynamic Load Allowance (DLA/FBD) for the KEL component of the "D" Load was taken as 40% for $L \leq 50$ m, while the DLA/FBD for the "T" Truck load was fixed at 30%. The "T" Truck axle configuration follows Figure 26 of SNI 1725:2016 (Figure 4): a 50 kN front axle is located at a fixed distance of 5.0 m from the 225 kN middle axle, while the distance between the middle and rear 225 kN axles varies from 4 to 9 m.

8.4.1 Besarnya pembebanan truk "T"



Gambar 26 - Pembebanan truk "T" (500 kN)

Figure 4. Configuration of the 500 kN "T" Truck load according to Figure 26 of SNI 1725:2016: 50 kN front axle and 225 kN middle and rear axles.

Source: Adapted by the author from SNI 1725:2016.

The transverse distribution factor (DF) was evaluated using the Lever Rule based on direct statics because the 5.0 m span lies outside the applicability range of the AASHTO LRFD stiffness-based approximate girder-distribution formulas, which require $L \geq 6000$ mm. Fourth, the Rating Factor (RF) was calculated using two approaches: a practical general equation with generic load factors and the official equation specified in Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016, which incorporates the condition factor (φc) and system factor (φs):

$$RF = [C - (\gamma DC)(DC) - (\gamma DW)(DW) \pm (\gamma P)(P)] / [(\gamma LL)(LL+IM)]$$

$$C = \varphi c \cdot \varphi s \cdot \varphi \cdot Rn, \text{ with the minimum requirement } \varphi c \cdot \varphi s \geq 0.85$$

The condition factor (φc) was assigned from visual observations of the girder condition ("substantially corroded but not yet damaging," corresponding to Condition Rating 2 in the guideline), while the system factor (φs) was selected from the guideline table for steel structures. RF was calculated for two failure modes: flexure at midspan and shear at the support.

Fifth, an additional independent verification was performed using a 2D grillage model, a stiffness-matrix finite-element method representing the bridge as interconnected longitudinal girder and transverse deck elements in accordance with the grillage-analogy principle. The model consisted of six longitudinal girder lines with nine stations along the span and transverse deck elements represented by ten mesh lines. Flexural stiffness (EI) was based on the corrosion-corrected composite section. The 2D grillage model is illustrated in Figure 5. As a higher-level independent verification, the bridge was also modeled in SAP2000 to obtain the maximum moments in the interior girders G3 and G4.

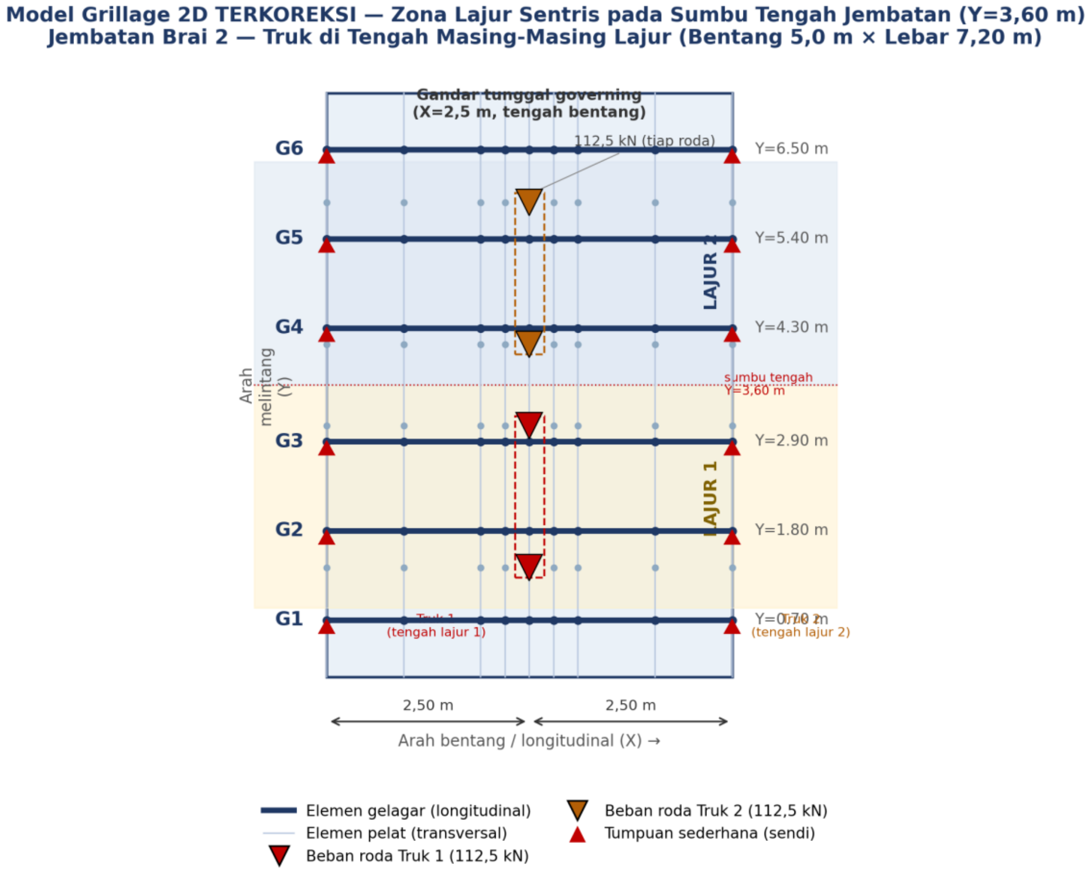


Figure 5. 2D grillage model showing the girder-deck mesh, wheel positions for the two-lane "T" Truck load, and simple supports at both ends of the span.

Source: Author modeling using Python, 2026.

2.5 Standards, Codes, and Technical References

The analysis was based on SNI 1725:2016, Bridge Loading, for traffic-load configurations and load factors; Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016, Guidelines for Determining Bridge Load Rating for Existing Bridges, for the Rating Factor procedure; the Albrecht and Naeemi (1984) model for atmospheric-corrosion prediction; ISO 9223:2012 for environmental-corrosivity classification; and the AASHTO LRFD Bridge Design Specifications for the Lever Rule and the applicability limits of girder-distribution formulas. SAP2000 was used as the finite-element verification tool.

3. RESULTS

3.1 Corrosion-Induced Thickness Loss and Section Reduction

For an effective unprotected exposure period of t = 56 years, the predicted thickness loss per exposed surface is:

$$C(56) = 70.6 \times 56^{0.79} \approx 1698\ \mu m \approx 1.70\ mm$$

Because the steel elements, including the flanges and web, are exposed to the atmosphere on both sides, the total thickness loss is $\Delta t = 2 \times 1.70 = 3.40$ mm, corresponding to approximately 17% of the original 20 mm section thickness. The corrected flange and web thicknesses are therefore tf' = tw' = 16.60 mm (1.66 cm), resulting in a corrected steel cross-sectional area of:

$$As' = 2(bf \times tf') + (d \times tw') = 2(14 \times 1.66) + (36 \times 1.66) = 106.24\ cm^2$$

This represents an approximately 17% reduction in steel cross-sectional area relative to the initial condition ($As = 128\ cm^2$).

3.2 Corrected Flexural and Shear Capacity

The plastic-neutral-axis check shows that the steel tensile force (T = 2549.8 kN) is lower than the maximum compressive capacity of the concrete slab (Cmax = 4675 kN). Consequently, the PNA lies within the concrete slab and the entire steel section reaches tensile yielding. The corrected nominal composite moment capacity is $Mn' \approx 973.5$ kN.m, a reduction of about 14% from the initial uncorroded capacity of $Mn \approx 1130$ kN.m. For shear, the web slenderness ratio h/tw = 21.7 is well below the limiting value of 71.0 (kv = 5.0, unstiffened web). The web is therefore classified as non-slender and can develop its full nominal shear capacity (Cv = 1.0) despite the thickness reduction, resulting in Vn = 860.5 kN.

3.3 Moments and Shear Forces Due to Traffic Loads

Evaluation of all "T" Truck axle-position combinations showed that the critical midspan flexural case occurs when a single 225 kN axle is located directly at midspan while the front axle and the companion 225 kN axle are outside the span. This condition produces a static maximum moment per lane of Mmax = 281.25 kN.m. The Lever Rule transverse distribution factor for the central girder G4 is DF = 0.678 under two-lane loading, giving a design moment from the "T" Truck load, including the 30% DLA/FBD, of 247.89 kN.m. This exceeds the design moment from the "D" Load of 125.27 kN.m. For support shear, the critical condition occurs when the middle and rear axles both remain on the span, resulting in a design shear force of 237.98 kN.

3.4 Rating Factor and Cross-Method Verification

The Rating Factors obtained from all analytical methods are summarized in Table 2.

Table 2. Summary of Rating Factors from all analytical methods

Flexure - General equation	0.678 (Lever Rule)	3.73	1.88
Flexure - PUPR Circular Letter 03/SE/M/2016	0.678 (Lever Rule)	3.51	1.77
Flexure - 2D grillage (GJ sensitivity)	0.447-0.471 (Grillage)	-	2.55-2.69
Flexure - SAP2000	Direct (FEM)	2.99	2.90
Shear - PUPR Circular Letter 03/SE/M/2016 (GOVERNING)	0.678 (Lever Rule)	3.89	1.64

Source: Author analysis, 2026.

The overall governing Rating Factor is 1.64, obtained from the support-shear evaluation under the "T" Truck load using the official procedure. This value is lower than the flexural RF of 1.77. Because both RF values remain ≥ 1.0, the Brai 2 Bridge superstructure formally satisfies the operational-feasibility requirement without load restriction in accordance with Section 5.1 of Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016.

For flexure, the Lever Rule DF of 0.678 is substantially more conservative than the grillage-derived DF range of 0.447-0.471. The lower grillage DFs produce considerably higher flexural RFs of 2.55-2.69, while SAP2000 gives an RF of 2.90, compared with 1.77 from the Lever Rule. The grillage range results from a sensitivity analysis of deck torsional stiffness (GJ): DF = 0.447 (RF = 2.69) is the best estimate using the calculated deck torsional stiffness, whereas DF = 0.471 (RF = 2.55) is a conservative scenario in which GJ is reduced toward zero. The maximum-moment and support-reaction distributions from the 2D grillage model across all six girders, which are perfectly symmetric about the bridge centerline, are shown in Figure 6.

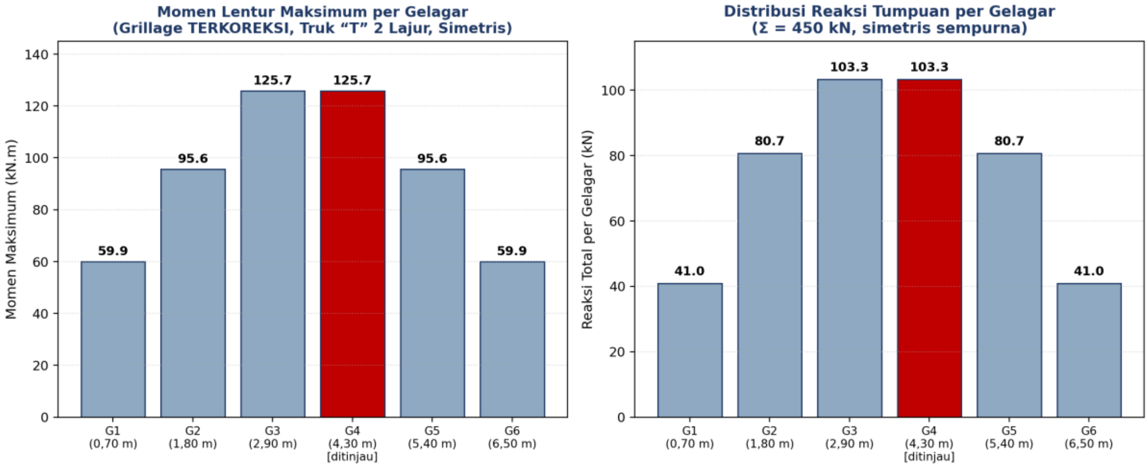


Figure 6. Maximum flexural-moment and support-reaction distributions across girders G1-G6 from the 2D grillage model, showing perfect symmetry (G1=G6, G2=G5, and G3=G4 as the critical girders).

Source: Author 2D grillage modeling, 2026.

Figure 7 compares the static moments, excluding DLA/FBD, produced by the two-lane "T" Truck load in all girders using the 2D grillage and SAP2000 models. For the governing girders G3/G4, the grillage result differs from SAP2000 by only about +7.8%.

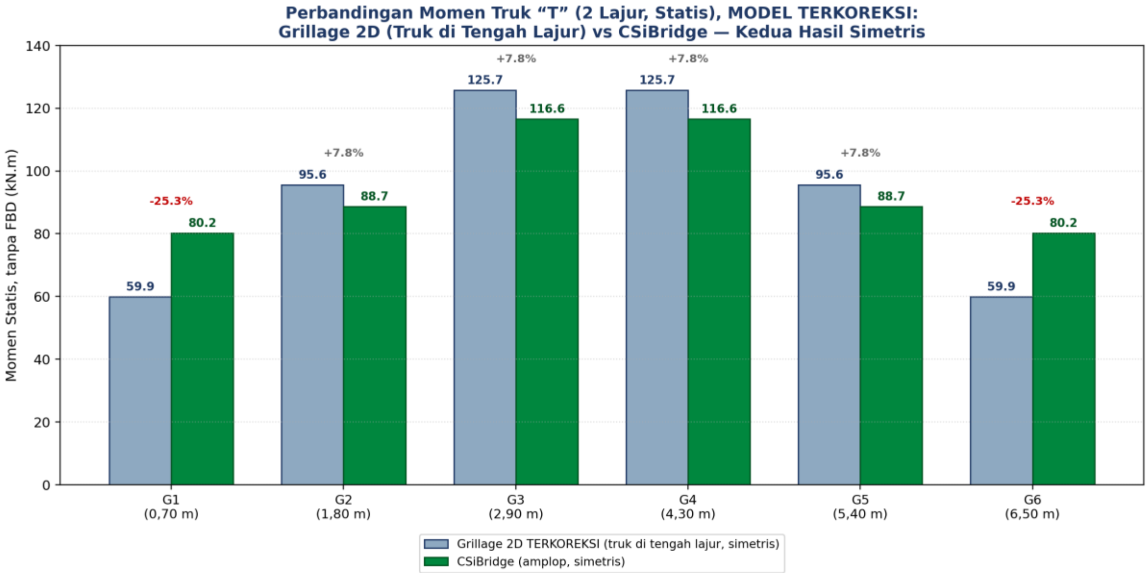


Figure 7. Comparison of static moments, excluding DLA/FBD, from the two-lane "T" Truck load in girders G1-G6: 2D grillage versus SAP2000.

Source: Author 2D grillage and SAP2000 modeling, 2026.

A similar pattern is obtained for the "D" Load. SAP2000 results in Figure 8 show a perfectly symmetric moment distribution across all six girders: G1=G6=98.7 kN.m, G2=G5=123.6 kN.m, and G3=G4=146.7 kN.m.

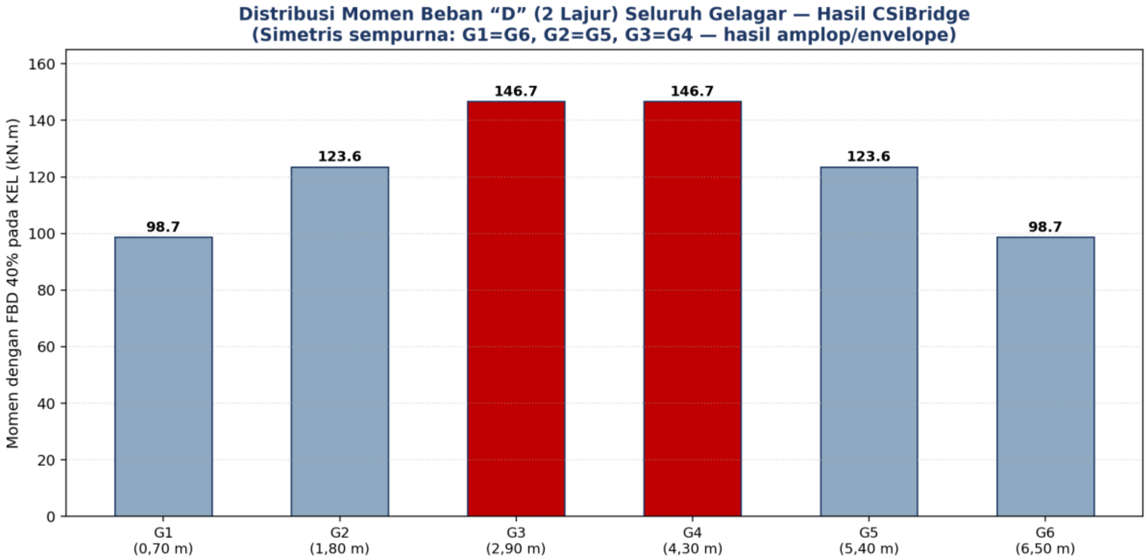


Figure 8. Distribution of moments from the two-lane "D" Load, including DLA/FBD, across all girders from SAP2000, showing perfect symmetry about the bridge centerline.

Source: Author SAP2000 modeling, 2026.

The detailed comparison of "D" Load moments between the 2D grillage and SAP2000 models in Figure 9 shows very close agreement for the governing girders G3/G4, with a difference of only approximately 1.9%, although the discrepancy remains larger for the exterior girders G1/G6.

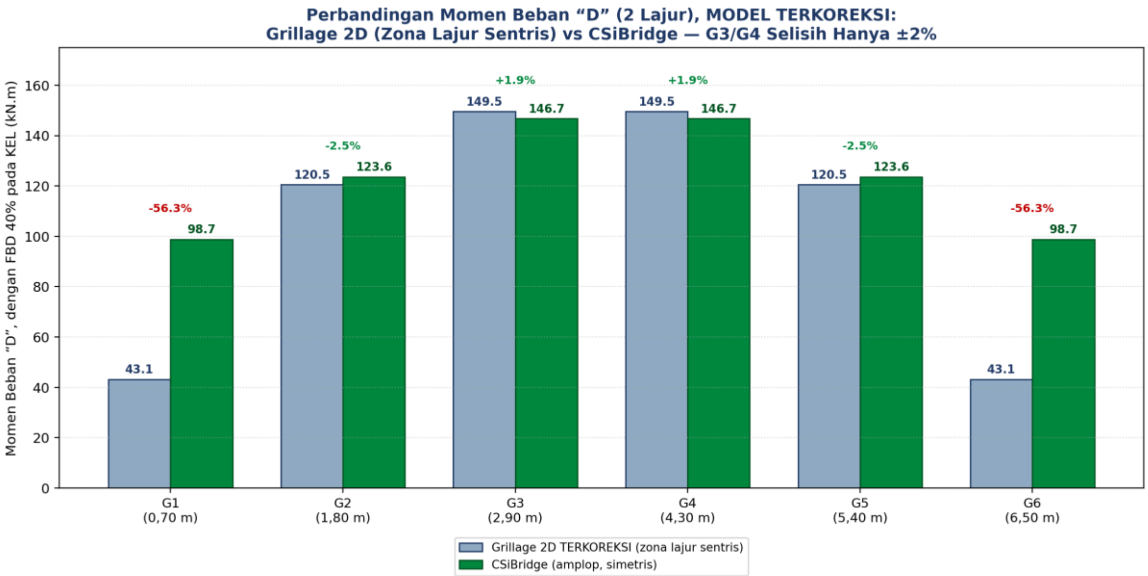


Figure 9. Comparison of "D" Load moments, including DLA/FBD, across all girders: 2D grillage versus SAP2000; the G3/G4 results are nearly coincident, with a difference of approximately 2%.

Source: Author 2D grillage and SAP2000 modeling, 2026.

4. DISCUSSION

The finding that shear, rather than flexure, governs the Brai 2 Bridge assessment indicates that the approximately 17% corrosion-induced reduction in girder-web thickness is more critical to shear resistance than to flexural resistance. This is consistent with the structural principle that uniform thickness reduction in a relatively thin web directly reduces the shear-resisting area, whereas composite flexural resistance is also strongly influenced by flange area and the internal lever arm. The finding has an important practical implication for inspection programs on similar bridges: the condition of girder webs near the supports should receive attention equal to, or greater than, that given to midspan flanges, which are often the principal focus of visual inspection.

Comparison of the three load-distribution approaches - Lever Rule, 2D grillage, and SAP2000 - shows that the Lever Rule is substantially conservative, producing a flexural RF of 1.77 compared with 2.55-2.69 from the 2D grillage verification and 2.90 from SAP2000. The close agreement between the independently developed grillage model and SAP2000, with differences of only about 2-8% for the governing girders, provides strong confidence that the actual flexural capacity margin is likely to be in the RF range of approximately 2.6-2.9 rather than at the conservative Lever Rule estimate. The larger differences for the exterior girders G1/G6 do not necessarily indicate modeling error. SAP2000 evaluates an envelope of possible transverse truck positions within a lane, whereas the independent grillage model represents only one truck position at the lane centerline. This difference should therefore be documented as a model limitation rather than treated as an analytical error.

The results are consistent with the general behavior reported in load-rating assessments of existing bridges, in which simplified load-distribution methods such as the Lever Rule tend to provide more conservative estimates than finite-element approaches, while still serving as a useful lower-bound safe estimate when advanced model verification is unavailable. The present study extends this methodological observation to a very short 5 m span that lies outside the applicability range of the standard AASHTO LRFD girder-distribution formulas, a bridge category that has received relatively little attention in load-rating studies.

From an asset-management perspective, RF ≥ 1.0 addresses the current static or ultimate capacity of the superstructure, but it does not by itself establish the adequacy of the entire bridge or guarantee its future serviceability. The calculated Rating Factor does not represent fatigue risk arising from repeated cyclic loading over several decades. This issue requires separate attention because the steel structure has been in service for 56 years and fatigue-detailing practices from the 1970s were generally less rigorous than modern provisions.

The study has four main limitations. First, the assessment does not include the substructure, namely abutments, piers, foundations, and bearings, which have different deterioration histories and failure mechanisms. Second, the corrosion-prediction model provides a screening-level estimate that has not yet been verified by direct field measurement using an ultrasonic thickness gauge. Third, the grillage model includes only one transverse truck-position scenario rather than the full load-position envelope represented in SAP2000. Fourth, fatigue risk has not been evaluated quantitatively. Future research should therefore combine direct thickness measurements, quantitative fatigue assessment of critical connections, and an integrated evaluation of both the superstructure and substructure.

5. CONCLUSION

This study concludes that, although Brai 2 Bridge has exceeded its intended service life, with 56 years of service compared with a nominal 50-year design life, and has experienced substantial coastal-corrosion-induced section loss of approximately 17%, corresponding to about 3.40 mm of thickness loss, the structural capacity of its superstructure remains adequate for the current traffic loads specified in SNI 1725:2016. The overall governing Rating Factor is 1.64 for support shear under the official procedure in Ministry of Public Works and Housing Circular Letter No. 03/SE/M/2016, lower than the flexural RF of 1.77. This confirms that the corrosion-thinned girder web is more critical in shear than the section is in flexure. The two-lane "T" Truck load governs over the "D" Load for this 5.0 m span in both flexure and shear.

Verification using the 2D grillage model, with RF ≈ 2.55-2.69, and SAP2000 finite-element modeling, with RF = 2.90, indicates a substantially larger flexural-capacity margin than the conservative Lever

Rule estimate of RF = 1.77. All calculated Rating Factors, ranging from 1.64 to 3.89, exceed the acceptance threshold of RF ≥ 1.0. Accordingly, the Brai 2 Bridge superstructure remains suitable for operation without load restriction, although its safety margin has been reduced by corrosion.

These findings have practical implications for the management of short-span bridge assets in Indonesia. They highlight the need to prioritize inspection of girder webs near supports rather than focusing only on midspan flanges; demonstrate the value of layered verification using manual calculations, a simplified grillage model, and finite-element modeling for bridges whose geometry lies outside the applicability limits of standard load-distribution formulas; and reinforce the need for periodic evaluation of aging short-span bridges that dominate many regency and rural road networks. Further work should include a separate load-rating assessment of the substructure, field verification of steel-section thickness, a dedicated fatigue-risk inspection and analysis, and medium- to long-term life-cycle planning for rehabilitation or replacement.

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AUTHOR CONTRIBUTIONS

Jampari Manukoa was solely responsible for the conceptualization, methodology, data collection and analysis, modeling including the 2D grillage and SAP2000 verification, and preparation of the manuscript.

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CONFLICT OF INTEREST STATEMENT

The author declares no conflict of interest regarding the publication of this article.

DATA AVAILABILITY STATEMENT

The data supporting the findings of this study are available from the corresponding author upon reasonable request.

ETHICAL STATEMENT

This study was conducted in accordance with academic research ethics. All data sources used in the study have been appropriately acknowledged. The research did not involve human participants or experimental animals.

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