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Performance Assessment of Reinforced Concrete Box Girder Bridge Under Legal and Permit Loads: A Case Study on Mille Bridge

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Abstract

Capacity assessment helps to identify the need of structures for repair, maintenance, preservation, reconstruction and replacement. The strength evaluation of many bridges in Ethiopia has been conducted on old and deteriorated bridges taking their super-structure part only. Performance of bridges must be assessed at early stage in order to know the initial performance the bridge and to ensure public safety and to protect public investment. In addition to this, assessment of sub-structure is helpful for preservation of the bridge's performance as it is one of the load transfer structures to the supporting soil. This study presents a comprehensive finite element-based methodology for the performance assessment of an existing reinforced concrete box girder bridge, the Mille River Bridge in Ethiopia, which is located on the Addis-Djibouti corridor. To assess nonlinear behavior under legal and permit loads, as well as seismic conditions, Finite Element Modeling (FEM) was conducted using ABAQUS/ CAE (for superstructure) and SeismoStruct (for substructure), respectively. Results indicate that the bridge possesses a significant reserve capacity, for both legal and the 190-ton permit loads. The pier's capacity-demand ratio was found to be above 1.20 for all load cases, indicating satisfactory performance under seismic demands. Despite satisfactory performance, maintenance of defects observed during field inspection and evaluation under flood-induced loads are recommended. The study concludes that the bridge can safely carry the current legal and investigated permit loads under investigation. Inspection of the bridge is recommended on regular basis.

Keywords: Box girder bridge, load rating, pushover analysis, FEM, performance assessment

1. Introduction

Bridges are critical infrastructures in road transportation networks, ensuring connectivity and economic growth. Their performance affects public safety and transportation costs directly. The effects of actual traffic loads on existing bridges often differ significantly from the design live loads due to variations in traffic volume, vehicle types, and weights (Nowak & Szerszen, 2003). Performance assessment of many bridges in Ethiopia has been conducted on old and deteriorated bridges taking their super-structure part only. However, in some cases, the condition of the substructure should be assessed accordingly. In addition to this, assessment of the substructure, particularly the pier system, is essential for ensuring the safety and long-term serviceability of the bridge. Consequently, maintenance and safety assessments based solely on original design capacities may lead to either overly conservative or not conservative decisions, resulting in uneconomical management or potential safety risks (Ghosh & Padgett, 2010). The load-carrying capacity of bridges tends to decrease over time due to factors such as overloading, accidental impact, and material deterioration. Simultaneously, increasing vehicle weights demand a higher capacity from aging structures (Akgul, 2004). While significant research exists on superstructure evaluation, the assessment of substructures, particularly piers, has received comparatively less attention, despite their critical role in overall bridge safety, especially under seismic events (Priestley et al., 1996).

In this study, focus has been given to Mille Bridge, a three-span reinforced concrete box girder bridge, and applies advanced finite element methods for superstructure assessment together with pushover analysis for pier evaluation were applied. The bridge under investigation is located on the Addis–Djibouti corridor, a heavily trafficked route essential for Ethiopia's import and export activities. It is also noted that the bridge exhibits adequate reserve capacities under legal and permit loads.

Unlike previous bridge assessment studies that focus primarily on either superstructure load rating (Gebre Y. et al., 2026; Hussein & Ali, 2020; and Bimrew, 2019) or substructure seismic evaluation (Taipe & Fernandez, 2023; Jaaz et al., 2021; Saadeghvaziri, & Yazdani, 2008; and Paulotto et al., 2007), this study presents an integrated framework combining detailed three-dimensional nonlinear finite element modeling of the bridge superstructure with performance-based seismic assessment of the substructure. The significance of this research lies in its practical application to bridge asset management where detailed bridge evaluation is often limited to visual inspection and simplified analytical methods.

2. Literature Review

The design and evaluation of bridges have evolved from Allowable Stress Design (ASD) to more rational methods like Load Factor Design (LFD) and Load and Resistance Factor Design (LRFD). The AASHTO LRFD specifications, first adopted in 1994, provide a probability-based approach to achieve a uniform level of reliability across different bridge types and loading conditions (AASHTO, 2017). These codes define four primary limit states: Service, Fatigue and Fracture, Strength, and Extreme Event. For existing bridges, evaluation is guided by manuals such as the AASHTO Manual for Bridge Evaluation (AASHTO, 2018), which introduces the Load and Resistance Factor Rating (LRFR) methodology. This approach mirrors the LRFD philosophy, allowing for a more realistic assessment of a bridge's safe load capacity using site-specific data (NCHRP, 2001).

Load rating methods such as Load Factor Rating (LFR) and Load and Resistance Factor Rating

(LRFR) provide systematic approaches to evaluate load carrying capacities (AASHTO, 2010; ERA, 2013). Pushover analysis has been widely adopted to study seismic performance, offering insights into structural capacity and ductility. Recent FHWA publications emphasize advances in integrating field inspection data and probabilistic methods to improve bridge evaluation (FHWA, 2004). Finite Element Modeling (FEM) is widely used for reinforced concrete box girders to simulate cracking, torsion, and nonlinear response (Agarwal et al., 2022; Zhang, 2021; Huang, 2022). Studies confirm that CDP-based FEM can replicate experimental shear and flexural behaviors (Wu, 2021; Huang, 2022) For seismic evaluation, pushover analysis provides capacity-demand insights with moderate computational effort. Researches demonstrate that pushover curves can effectively identify hinge formation and displacement capacity (Bergami et al., 2020; Calvi et al., 2020). Recent works extend pushover and fragility methods to consider multi-hazard conditions, including floods and earthquakes (Gidaris & Padgett, 2021; Xu et al., 2024; Zhang, 2024).

Recent studies have demonstrated the increasing application of advanced finite element methods and performance-based assessment techniques for evaluating existing bridge structures. Nonlinear finite element modeling using the Concrete Damaged Plasticity (CDP) model was employed to accurately simulate the damage behavior of reinforced concrete bridge piers subjected to monotonic loading, demonstrating the capability of numerical models to capture cracking, stiffness degradation, and ultimate failure mechanisms (Ahmad et al., 2023).

Dakhili et al. (2024) proposed automated approaches for damage assessment of prestressed concrete bridge girders using deformation-based indicators, demonstrating the growing role of numerical methods in bridge asset management. Similarly, the study by Santos et al. (2024) used finite element model updating techniques to detect damage and assess the condition of existing bridges, highlighting how crucial calibrated numerical models are for obtaining reliable structural evaluations.

In the area of seismic assessment, recent investigations have highlighted the effectiveness of nonlinear pushover analysis for evaluating bridge performance under extreme loading conditions. Bakalis et al. (2024) integrated monitoring data with pushover analysis to identify seismic damage in reinforced concrete bridges and demonstrated the applicability of performance-based assessment approaches. In addition, a recent study showed that nonlinear performance assessment methods provide valuable insight into damage progression, displacement capacity, and bridge resilience under seismic actions (Liu et al., 2024).

While recent advances have emphasized on life-cycle assessment and reliability-based evaluation, most existing research tends to focus on either the superstructure or the substructure separately, rather than considering the bridge as an integrated system. Biondini & Frangopol (2020) apply probabilistic deterioration models, while Pan & Lu (2023) propose hybrid approaches combining FEM and field measurements. These advances highlight the need for integrated methodologies that can be adapted in Ethiopia for critical corridors like Addis–Djibouti, where traffic volumes and overloading rates are significantly high (Gebre, A., 2025). While numerous studies have focused on superstructure rating, integrated assessments combining detailed FEM analysis of superstructures and nonlinear seismic analysis of substructures for specific bridges are limited. This study fills this gap by conducting a detailed 3D FEM analysis of a box girder bridge

superstructure in Abaqus, performing a nonlinear pushover analysis of the bridge pier in SeismoStruct. This combined approach allows for a comprehensive evaluation of bridge safety and performance under both legal and permit loads. The findings also offer practical, recommendations for bridge management authorities.

3. Materials and Methods

3.1 Description of the Mille River Bridge

Mille Bridge (A1-7-009) is a three-span reinforced concrete box girder bridge with a span length of 31 m each, supported on 9.76m high reinforced concrete piers with elastomeric bearings and masonry abutments as shown in Figure 1. It is a critical crossing over the Mille River Gorge on the Addis–Djibouti corridor, which is 521.3 km from the Addis Ababa situated 499m above the mean sea level, and carries heavy freight traffic. Field inspections recorded surface deterioration and local drainage clogging.



Figure 1: Mille Bridge

3.2 Finite Element Modeling and Validation

Since direct field load testing of the Mille Bridge was not available for model calibration, in this study, validation was performed using experimentally tested reinforced concrete beams and columns reported in the literature. Although the validation demonstrates the capability of the adopted modeling techniques to reproduce reinforced concrete behavior, direct validation of the bridge model remains a limitation of the present study.

3.2.1 Validation of Reinforced Concrete Beam

Studies have demonstrated the efficacy of ABAQUS in reliably predicting the response of reinforced concrete beams and bridges (Genikomsou & Polak, 2015; Muzeyin, 2016). A reinforced concrete beam tested by Muzeyin (2016) was modeled in ABAQUS to validate the material models and modeling techniques. The beam was simply supported with a concentrated load at mid-span. Concrete was modeled using 8-node brick elements (C3D8R) with the Concrete Damaged Plasticity model. Reinforcing steel was modeled as 2-node truss elements (T3D2) and embedded within the concrete mesh.

The beam specimen considered for validation is a rectangular cross section of b x h= 250 x 200mm, with overall length of 1700mm and 1300mm distance between supports was prepared. Three deformed bars on the bottom surface with 16mm in diameter and two deformed bars on top surface with 8 mm in diameter were provided. For the stirrups, 6 mm diameter deformed bars with a spacing of 50 to 800 mm were used. A concrete cover of 25 mm was provided in all surfaces of the beam. Concrete with a 28 days characteristic compressive strength, f'_c of 14.79 MPa and steel bar with yield strength of 682.52 and 720.74 MPa were used. For testing, a point load was applied at the midspan of the beam and the experimental result is compared with the FEM result.

The non-linear finite element software package, ABAQUS, is used for simulation. The simulation of a reinforced concrete beam requires an accurate model of the structural elements and its constituent members acting as a composite made up of concrete and steel. In order to

develop concrete beam, 8- node continuum solid elements were utilized. The adopted CDP parameters were selected based on recommendations available in the literature (Ahmad et al., 2023; and Bimrew, 2019) for normal-strength reinforced concrete. CDP assumes that the main two failure mechanisms are tensile cracking and compressive crushing of the concrete material. The dilation angle was taken as 33°, eccentricity as 0.1, f_{b0}/f_{c0} ratio as 1.16, K_c as 0.667, and viscosity parameter as 0. Reinforcement was modeled using embedded truss elements assuming perfect bond between steel and concrete.

A mesh sensitivity study was conducted to ensure that the numerical results were independent of mesh discretization. Several mesh sizes were investigated and results indicated that further mesh refinement beyond the selected mesh size produced negligible changes in the response, confirming mesh convergence and ensuring computational efficiency. Size of elements in the mesh was defined to be 150mm. Figure 2 shows a three-dimensional view of the model, which was used to develop the concrete beam. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing.

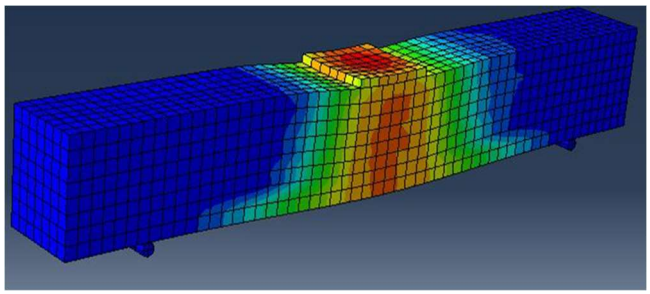


Figure 2: Deformed shape of the RC beam

Validation against experimental data confirms the reliability of the numerical models. Load - displacement values measured at the center of the beam from the experiment results (Muzeyin, 2016) and the present finite element modelling are shown in Figure 3. The results showed excellent agreement between the FE model and the load-displacement curve of the experimental beam, confirming the accuracy of the chosen modeling approach.

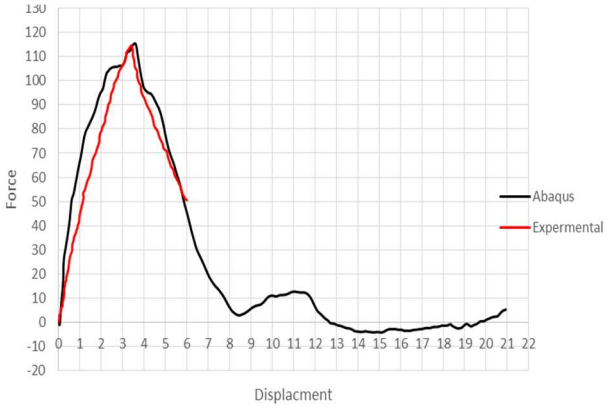


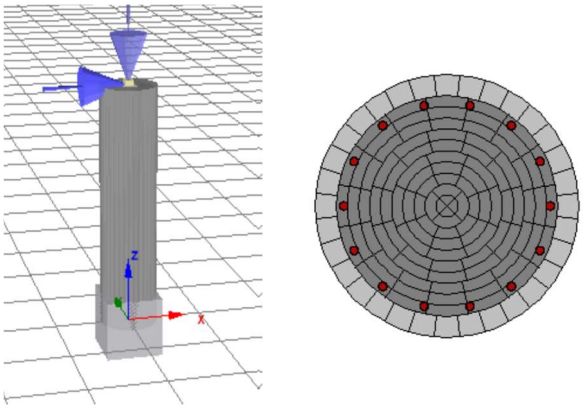
Figure 3: Comparison of load-displacement diagrams of the experimental tested beam with FE analysis

3.2.2 Validation of Reinforced Concrete Pier

Performance-based earthquake engineering frameworks assess piers based on predefined performance levels by comparing displacement demands against capacities (FEMA, 2000). The seismic performance of bridge piers is commonly evaluated using nonlinear static (pushover) analysis. This method involves applying a monotonically increasing lateral load to the structure until a target displacement is reached, revealing the sequence of yielding, plastic hinge formation, and the ultimate capacity (Fajfar, 2000). Fiber-based modeling, as implemented in SeismoStruct, discretizes the pier section into fibers with specific material properties, providing an accurate representation of its nonlinear behavior under combined axial and flexural loads (SeismoSoft, 2020).

The column considered for validation is a circular, spiral-reinforced concrete column tested in a cantilever configuration (Lehman et al. 2001). The column has a diameter of 609.6 mm and a length of 2438.4 mm, cast with 31

MPa concrete. It is reinforced longitudinally with twenty-two 15.9 mm bars having a yield stress of 462 MPa (ultimate strength 630 MPa), and laterally confined using a 6.4 mm diameter spiral spaced at a 31.8 mm with a cover of 22.2 mm to the center of the spiral. The transverse reinforcement yield stress is 606.8 MPa. During testing, the column was subjected to a constant axial load of 653.86 kN. Figure 4 illustrates a loaded pier modeled with discrete segments and reinforcing bars. Figure 5 presents a comparison between the laboratory test results and the finite element (FE) analysis, demonstrating good agreement and confirming that the numerical results are consistent with the experimental observations.



a) b)

Figure 4: a) pier loading and b) discrete segments of the pier with reinforcing bars

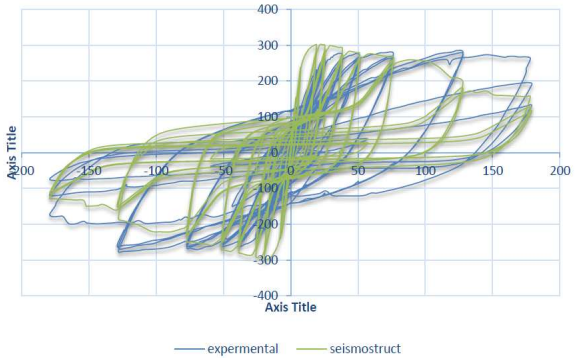
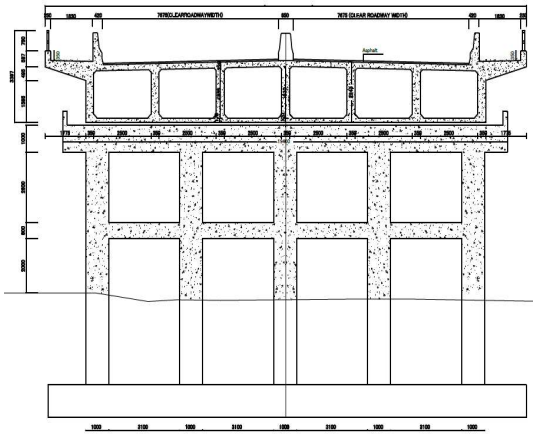


Figure 5: Comparison of laboratory tested column with FE analysis

3.3 Bridge Data and Modeling

3.3.1 Bridge Data

The Mille's bridge superstructure is composed of a six-cell box girder system, with its primary supporting girders spaced at 2,800 mm on center. The girder has a total depth of 2,243 mm, 350 mm web thickness, a 220 mm thick top deck slab, and a 170 mm thick bottom flange. Reinforcement within the girders is detailed for both flexural and shear capacity: the webs are reinforced with 16 mm diameter bars at 200 mm spacing as skin reinforcement, while the main flexural resistance is provided by 32 mm diameter bars, 30 bars in each interior girder and 20 bars in each exterior girder, distributed across their effective widths. In shear critical zones, 12 mm diameter stirrups are densely placed at 80 mm intervals. Cross-sectional and reinforcement detailing for both interior and exterior girders are illustrated in Figure 6. The substructure consists of reinforced concrete piers and they are spaced at 4,550 mm on center. Each pier is a 1,000 mm diameter circular column, longitudinally reinforced with 16 bars of 32 mm diameter and laterally confined by 12 mm diameter stirrups at 200 mm spacing. To ensure lateral stability, the pier is braced at 3,500 mm from its top by a 600 mm square beam reinforced with 10 ϕ 20 bars distributed at the top and bottom surfaces.



a)

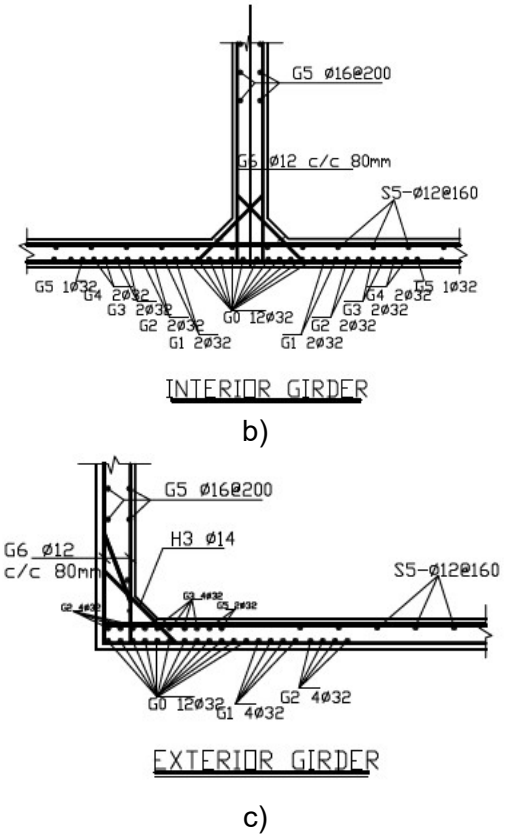


Figure 6: Mille bridge a) cross section b) interior girder reinforcement and c) exterior girder reinforcement details

3.3.2 Superstructure Modelling

The superstructure was modeled in ABAQUS/CAE 6.19-1 using 3D eight-node solid elements for concrete and truss elements for reinforcement. CDP was adopted to capture tensile cracking and compressive crushing.

3.3.3 Material Properties

Material properties used in the FEM model is shown in Table 1.

Table 1: Material properties of concrete and reinforcement bar

Material properties	reinforcement bar	concrete
Initial Stiffness GPa)	200	29
Compressive strength (MPa)	-	25
Yield strength (MPa)	420	
Poisson's ratio	0.3	0.2
Unit weight (kN/m^3)	78	24

3.3.4 Boundary Conditions

Due to the geometric symmetry of the bridge considered in the analysis, only half of the bridge model was simulated to reduce computational demand. Symmetry boundary conditions were applied along the longitudinal plane of symmetry by restraining displacement normal to the symmetry plane and the associated rotational degrees of freedom. The validity of this assumption was verified by ensuring that the applied loading configuration remained symmetric with respect to the bridge centerline.

3.3.5 Load Cases

The live loads are selected as per ERA's bridge design manual (ERA, 2013) recommendation. The four Legal Trucks and permit truck is applied on the deck for calculating maximum nominal live load effects. The axle configurations and axle weights of the legal load and permit load (190 tons), used for the analysis, are shown in Figures 7 and 8, respectively. While applying the loads to get maximum effect one Legal Truck has been considered. Since action effects are sensitive to the positions of the assessment loads, different positions of the legal loads (both transverse and longitudinal) are considered to determine the maximum effect.

The dimensions for tire contact area used in the analysis is as per ERA's bridge design manual (ERA, 2013). The three-dimensional finite element model (3D FEM) of the bridge structure with vehicle contact area and load positions for different loading arrangements are shown in Figure 9. As shown in Figure 9, for legal vehicle loads, a rectangular contact area of 240 mm in length and 375 mm in width was applied. For the permit vehicles, a larger contact area of 250 mm by 510 mm was used.

To determine the most critical structural response, several transverse and longitudinal positions of the legal and permit vehicles were

investigated. Vehicle locations were systematically varied to identify the loading arrangement producing the maximum bending moments and shear forces. The governing load position corresponding to the maximum response was subsequently adopted for load rating calculations.

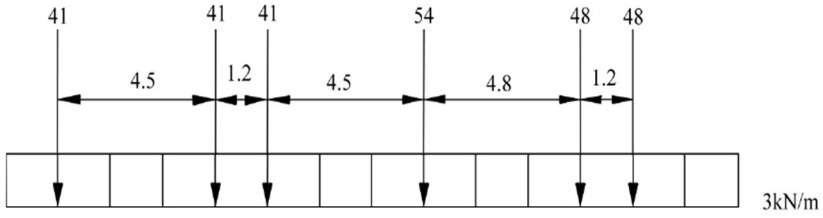


Figure 7: The legal lane loading (mainly for large spans)

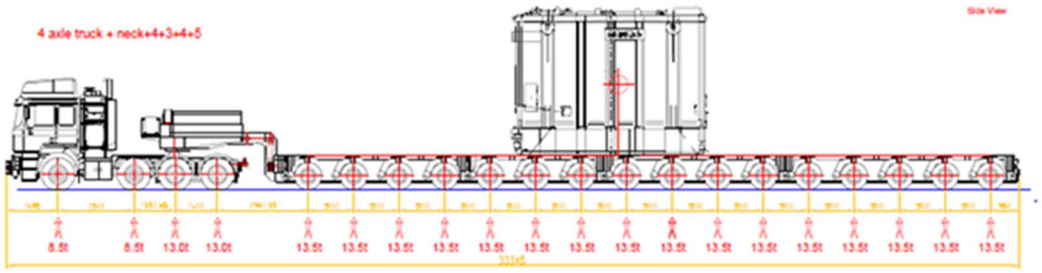
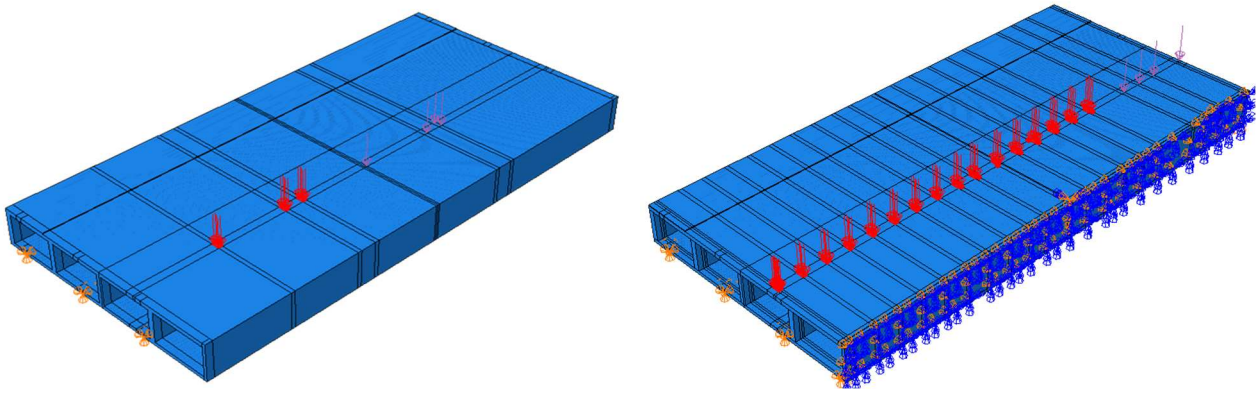


Figure 8: 16 axle trailer truck for permit loads (190-ton)



(a) (b)

Figure 9: Vehicle to bridge contact a) legal load and b) permit load

3.4 Substructure Modeling

The pier was modeled in SeismoStruct 2020 with force-based fiber elements representing circular cross-sections, including axial load effects from the superstructure. The target displacement was selected as 4% of the pier height based on performance-based seismic assessment recommendations and previous studies on reinforced concrete bridge piers. The selected value allows adequate characterization of yielding, plastic hinge development, and post-

yield behavior while remaining within realistic deformation limits. Performance levels were evaluated according to displacement capacity and damage states associated with operational and life-safety performance objectives. The 3D model of the pier in SeismoStruct is shown in Figure 10.

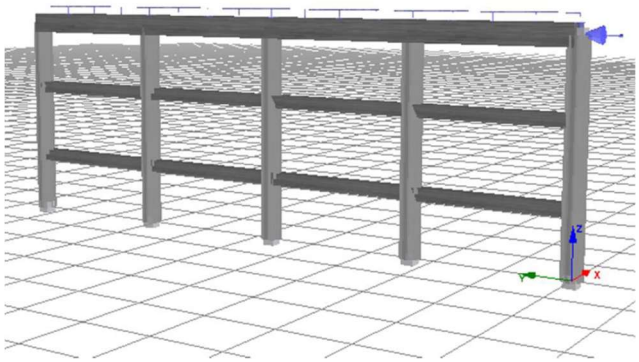


Figure 10: Pier elevation

4. Results and Discussion

The results indicate that the critical structural responses occur near the mid-span region where maximum positive bending moments develop and at support locations where shear demands are highest. The observed reserve capacity demonstrates that the bridge possesses significant redundancy beyond current legal loading requirements. Under permit loading conditions, stress redistribution within the box girder system contributes to efficient load transfer among adjacent webs, thereby limiting localized overstressing. The relatively high rating factors indicate that the structure was designed with considerable safety margins and currently exhibits satisfactory structural performance.

4.1 Superstructure Response

Maximum mid-span deflections and support reactions were extracted from Abaqus. The maximum mid-span displacement under permit loading reached the elastic-plastic transition but did not show global instability. The displacement contour of the bridge under legal load is shown in Figure 11.

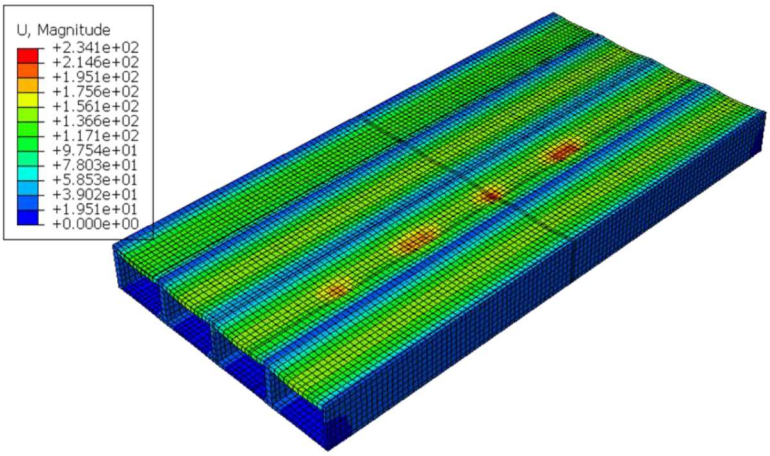


Figure 11: Displacement contour

4.2 Load Rating Results

The bridge analyses were performed using the legal loads. The applied load consists of pressure loads, impact load, and dead load applied to deck elements. Load-ratings of bridges were computed using the ERA BDM (2013) and AASHTO Manual for Bridge Evaluation (AASHTO, 2018).

Results showed that the bridge possesses considerable reserve capacity under legal loads (rating factor of 3.06 for bending) and permit loads (rating factor of 1.74 for shear), as shown

in Table 2, which are significantly greater than the acceptance criterion of 1.0 specified in codes and standards (AASHTO, 2018; and ERA, 2013), indicating that the bridge can safely accommodate the applied loads (Bimrew, 2019 and NCHRP Report 292, 1987).

Table 2: Bridge Load Rating Computation

Load Case	Section	Shear (kN)	Moment (kN-m)	Rating Factor
Legal Load	Right end support / mid-span	188.47	1824.55	3.06
Permit Load	Right end support / near mid-span	630.57	5318.29	1.74

All rating factors are significantly greater than 1.0, indicating that the bridge has a better reserve capacity and can safely carry the current traffic and the specific 190-ton permit vehicle without requiring strengthening. The obtained results are consistent with previous bridge evaluation studies (Sediek & Darwish, 2026) that reported rating factors greater than 1 for structurally sound bridges. Hussein and Ali (2021), in their load rating assessment of Al-Halfaia Bridge in Sudan, concluded that the bridge possessed adequate reserve capacity under legal and permit loading scenarios when evaluated according to AASHTO procedures.

4.3 Pier Pushover Results

The non-linear pushover analysis yields a capacity curve and a capacity-demand ratio exceeding one for, legal and permit load cases (Table 3). Base shear to displacement curve and deformed shape of RC pier under the given loading conditions are shown in Figures 12 and 13, respectively. A capacity-demand ratio greater than 1.2 for both legal and permit loads has been obtained as shown in Table 4. The obtained results are consistent with previous studies on reinforced concrete bridge piers assessed using nonlinear pushover analysis (Taipe & Fernandez, 2023; and Saadeghvaziri, & Yazdani, 2008).

In performance-based bridge assessment, a capacity-demand ratio greater than 1.0 generally indicates that the structural component has sufficient capacity to resist the applied demands without reaching its limit state (Jaaz et al., 2021).

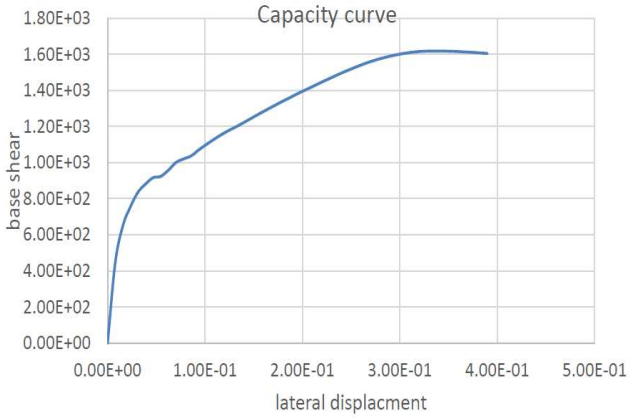


Figure 12: Base shear – displacement curve

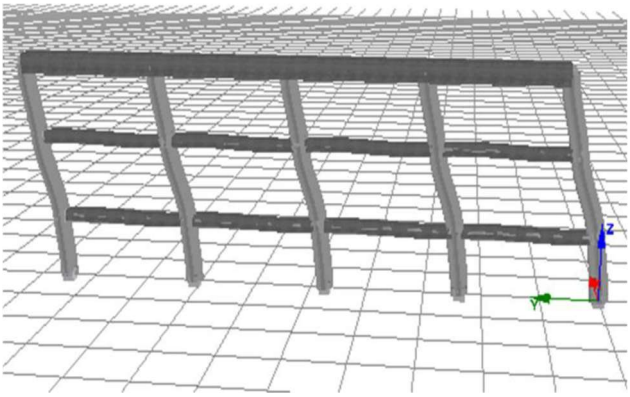


Figure 13: Deformed shape of RC pier

Table 3: Pier Capacity-Demand Ratio

Load Case	Capacity (kN)	Demand (kN)	Capacity/ Demand
Legal Load	1599.05	1300.46	1.23
Permit Load	1740.94	1240.14	1.40

5. Conclusions

This study applies an integrated FEM and non-linear pushover-based approach to assess the performance of Mille Bridge. Based on the study, the following conclusions are made:

- The finite element analysis indicates that the superstructure possesses substantial reserve capacity under both legal and investigated permit loading conditions. The calculated rating factors exceed 1.0 by a significant margin, demonstrating that the bridge can safely accommodate the current traffic demands and the investigated 190-ton permit vehicle without requiring structural strengthening.
- The superstructure components of the bridge exhibit substantial reserve capacity under both legal and permit loads which corresponds to a reserve capacity exceeding 74%.
- pushover analysis indicates that the pier satisfies the Fully Operational performance level, with no immediate need for retrofitting.
- The integrated assessment framework developed in this study provides a practical methodology for evaluating existing bridges and supports evidence-based decisions regarding maintenance planning, load permitting, and long-term infrastructure management.

Future studies should incorporate field monitoring data, probabilistic assessment techniques, and advanced dynamic analyses to further improve the reliability of bridge performance assessment. Moreover, regular inspection for maintenance intervention and implementation of continuous monitoring for long-term performance management are recommended.

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