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Evaluation of Curved Steel Tub Girder in Elevated Metro Rail Structures: Load Analysis and Design Optimization

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ABSTRACT: This paper presents a comprehensive study on the construction stage analysis of a tub girder superstructure for an elevated metro rail bridge with a span length of 60 meters and a plan radius of 225 meters. The study addresses critical challenges associated with constructing long-span curved bridges, particularly focusing on the stability during erection, the sequencing of deck pouring, and deflection control to ensure passenger comfort. A grillage model was developed using MIDAS Civil to simulate the construction stages, including the impact of different sequence strategies on the superstructure's stability and performance. The findings reveal that the construction process can be carried out effectively without disrupting traffic flow by carefully planning the sequencing of activities. Additionally, the design of the superstructure is primarily governed by deflection limits rather than stress limits, ensuring the structure remains within serviceability requirements during operation. Key improvements were observed in the stability of the inner bearing, reduction in compression stresses, and enhanced load distribution through the use of external bracings and diaphragms. The study also highlights the importance of torsional resistance, which is significantly improved with the use of box diaphragms. The results underscore the effectiveness of optimizing construction stages to enhance the stability, safety, and performance of the bridge structure during both construction and service stages. The insights gained from this study provide valuable guidance for the design and construction of elevated metro rail bridges, particularly those with sharp curvatures and long spans.

KEYWORDS: Tub Girder, Elevated Metro Rail, Construction Stage Analysis, Deflection Control, Grillage Model, Curved Bridge, Torsional Resistance.

I. INTRODUCTION

The design and construction of bridges for metro rail systems, particularly those involving long spans and curved alignments, present unique challenges. Elevated metro rail systems are essential for modern urban transport, especially in densely populated cities where space constraints and the need for efficient transportation solutions require the use of elevated structures. Among the various types of bridge superstructures, tub girder composite systems are increasingly preferred due to their superior torsional resistance and structural efficiency, especially for curved alignments. These bridges often involve spans that exceed conventional lengths and require intricate construction techniques to maintain safety, stability, and serviceability. A significant challenge in the construction of curved steel tub girder bridges is managing the complex behavior that arises during construction and service stages. Horizontal curvature introduces torsional, bending, and distortional effects on the structure, requiring careful attention to the construction sequence and the impact of loads during the erection phase. In addition, large span lengths introduce issues related to stability during the casting of the concrete deck and the eventual load distribution when the bridge is in service.

Metro rail bridges at major junctions often require obligatory spans with sharp curvatures, where conventional bridge construction methods may not be feasible. Typically, precast segmental superstructures are used for standard spans of up to 35 meters, but at these junctions, spans can extend up to 60 meters or more. This increased span, combined with sharp curvatures, presents challenges in both the selection of the superstructure and the execution of construction within busy urban environments. Ensuring traffic movement and minimizing disruption while constructing these superstructures becomes a critical aspect of the project. To address these challenges, this paper aims to explore the construction stage analysis of a tub girder composite superstructure for a typical 60-meter span curved bridge, considering factors such as deflection control, stability during erection, and optimum construction sequencing. The study uses MIDAS Civil software to simulate the construction stages and assess the stability and behavior of the structure under different loading conditions. This paper focuses on analyzing the impact of the erection sequence, the

pouring of the deck concrete, and the structural performance during the service stage, aiming to provide design strategies to mitigate issues like torsion, stress concentration, and deflection.

This work is particularly significant given the limited focus on construction stage analysis for tub girder superstructures, particularly in the context of large, curved spans. By providing insights into the construction challenges and behavioral analysis, this study aims to contribute to the safe and efficient construction of elevated metro rail systems, ensuring that they meet both engineering standards and operational requirements for passenger comfort and long-term durability..

II. LITERATURE SURVEY

The literature survey delves into the advancements and trends in the analysis, design, and construction of curved steel tub girders for elevated metro rail systems. It highlights both theoretical advancements and practical applications to address the challenges posed by complex geometries and dynamic loadings in metro rail bridge construction. The survey identifies key research areas, especially the improvements needed in Finite Element Method (FEM) models, as well as optimization strategies in materials and design processes aimed at enhancing sustainability and performance.

In the late 1960s, the Consortium of University Research Teams (CURT) was formed to study the behavior of horizontally curved bridges in the United States. This research led to the development of analytical and experimental techniques to understand the behavior of curved steel bridges, although the guide specifications from the CURT project were not integrated into the AASHTO Standard Specification for Highway Bridges. Later, in 1992, the Federal Highway Administration, in collaboration with 13 states, launched the Curved Steel Bridge Research Project to further investigate the performance of curved steel bridges, focusing on theoretical, analytical, and experimental methods.

Charles W. Roeder *et al* performed experiments on various bearing types (pot, spherical, and disk bearings) for curved steel bridges. The results highlighted the advantages and potential issues related to each bearing type, such as leakage in pot bearings and uplift concerns in disk bearings. Further, James S. Davidson and Chai H. Yoo analyzed the effects of curvature on the bending strength of curved I-girders using finite-element models. Their research demonstrated the accuracy of predictor equations for evaluating the behavior of curved members and identified areas for improvement in existing models. Zhanfei Fan and Todd A. Helwig focused on torsional loads and brace forces in steel box girders, with an emphasis on the distortional components of torsional loads. They provided equations for brace forces in quasi-closed box girders.

Nam-Hoi Park *et al.* conducted parametric studies on the spacing of intermediate diaphragms in curved box girder bridges. Their findings proposed new design charts that offer more flexibility in diaphragm spacing based on desired stress ratios, providing a better design approach than current practices. Magdy Samaan *et al.* [5] performed a parametric study of curved box-girder bridges using the finite-element method. The study explored various parameters such as span-to-radius ratio and truck loading types to derive empirical formulas for load distribution factors. D. R. Popp and E. B. Williamson highlighted construction issues, such as lateral torsional buckling in trapezoidal box-girder bridges, and developed software to investigate the behavior of steel box girders during the construction phase, emphasizing the importance of considering partially-composite behavior. Kyungsik Kim and Chai H. Yoo studied the interaction between top lateral and internal bracing systems in tub girders. They identified significant coupling actions between bracing members and internal cross-frame members, which influence the stability and strength of the structure during construction.

B. J. Bell and D. G. Linzell used the SAP 2000 software to model a three-span continuous bridge, investigating the effects of erection sequencing, lateral bracing, and temporary supports on the deformed shape of the curved superstructure. They found that paired girder erection and the use of lateral bracing effectively reduced deflections. Chai H. Yoo *et al.* developed interaction equations for predicting the bending strength of curved I-girders subjected to combined vertical bending and torsion. Their study emphasized the limitations of non-compact sections and introduced compact-flange sections as a practical solution for improving strength and capacity. Kuppumanikandan A. carried out a parametric study using FEM to investigate the behavior of single, double, and triple-cell box girder bridges. The study showed that as curvature decreases and span length increases, the longitudinal stresses, shear, torsion, and deflections also increase. Kevin D. Sear and Susan Steele conducted a case study of long-span steel bridges, focusing on stability during erection and concrete deck placement. Their findings highlighted the importance of staged concrete placement and lateral bracing in maintaining stability during construction. C. Tony Hunley and Issam E. Harik analyzed the

redundancy of twin steel tub girder bridges through nonlinear finite-element analyses. They found that twin girder designs, if properly proportioned, can be redundant and provide enhanced load transfer from a damaged girder to an undamaged one.

The literature highlights the complex behavior of curved steel tub girders, emphasizing the need for advanced modeling techniques, efficient material usage, and careful construction planning to address challenges such as torsion, deflection, and overall stability. The findings also point to the importance of using FEM for analyzing these structures and suggest areas for further research, especially in the context of large, curved spans and metro rail systems.

The construction of superstructures for metro rail systems, particularly in areas with sharp curvatures and long spans, presents significant challenges, especially in the erection process and deck pouring sequencing. There is limited research on the construction stage analysis of tub girders, which are commonly used in such structures. This study aims to address these challenges by analyzing a 60m long span with a radius of 225m, which is typical for major junctions in metro systems. The research investigates the use of a tub girder composite superstructure, focusing on its stability during various construction stages, including the erection sequence and concrete pouring. The study uses grillage analysis with construction stages in MIDAS software to assess the behavior of the superstructure under metro loading, including parameters such as bending stress, deflection, and fatigue. The research also aims to optimize the superstructure's design and explore the selection of appropriate bridge articulation and bearing types for the span. The findings of this study will contribute to improving the design and execution of metro rail viaducts with long spans and sharp curvatures, ensuring passenger comfort and structural integrity.

III. METHODOLOGY

For the analysis purpose, an obligatory span curved in plan is selected from the proposed alignment of a metro line. As the alignment crosses the junction, it is mandatory to keep a clear roadway for the below traffic. It can be seen that the alignment crosses the junction diagonally near the corner that is why a span length of 60m has been proposed. Details of curved alignment for 60m span are as follows, circular curve length of 60 m with radius of 225m.

1) Configuration of metro bridge

The bridge is proposed to cater for two lane metro rail. Total width of proposed bridge is 8.50m with two tracks for up and down lane and parapet walls at either edge. The rail is supported on concrete plinth running throughout. Deck of the bridge is kept sloping inwards from both edges at a slope of 2.5% to collect the rain water for disposal. For a span of 60m choice of superstructure depends on various aspects. For majority of the alignment precast segmental box girder is proposed with standard spans upto 35m. In this case maximum length of the span is governed by the length of the launcher used to lift the segments; therefore use of this type of superstructure is not feasible for a span of 60m. Also cast in situ superstructure is not feasible as it will require all the space below to erect staging for its construction. Therefore steel superstructure is the best option as it will be fabricated in the yard and it will be erected in position without disturbing the traffic below.

2) Design Optimization and Model Comparison

In this study, three different models are prepared and analyzed to compare their performance under various loadings. The comparison is based on key design parameters such as bending stress, deflection, and reaction. All models share similar design aspects, except for the differences outlined below, which affect the structural response.

Model-1: Single Support at the Centreline of Each Girder

In **Model-1**, a single support is provided under each girder, placed along the centerline of the girder. This model serves as the baseline for comparison, where the bending stress, deflection, and reaction are calculated based on the girder's properties under typical loading conditions. The centerline support configuration represents a simple and traditional support arrangement that does not account for the effects of curvature or temporary support systems.

Model-2: Shifted Supports and No Intermediate Temporary Supports. **Model-2** involves shifting the positions of the supports towards the outer side of the curvature by 0.4m, resulting in a more efficient load distribution. This configuration also considers a girder length of 60m to be erected on bearings, without any intermediate temporary supports. The bending stresses are calculated for the self-weight of the girder and the wet concrete based on the girder-only section properties. This model aims to evaluate the effects of support positioning and the absence of intermediate supports on bending stresses, deflections, and reactions.

Model-3: Shifted Supports with Temporary Supports and 36m Maximum Span. In *Model-3*, the supports are again shifted 0.4m outward from the centerline of the curvature, similar to *Model-2*. However, this model incorporates intermediate temporary supports and limits the maximum span to 36m during construction. Temporary supports are used during the erection process and are removed once the girder section becomes composite (after 28 days of casting the deck slab). The bending stresses for the self-weight of the girder and wet concrete are calculated using the girder-only section properties for the construction stage with a 36m maximum span. For the final stage, the bending stresses due to Dead Load (D.L), SIDL, and Live Load are calculated using composite section properties, which more accurately reflect the completed structure.

3) Construction Stages for Model-3

Model-3's construction stages depend on the erection scheme and the pouring sequence of the deck concrete. When planning the erection scheme, several factors must be considered, including the required clear roadway for traffic and the maximum crane capacity available to lift the girder segments. Based on these considerations, the 60m girder length is divided into five equal segments of 12m each. The middle three segments will form a girder length of 36m, which can be temporarily supported over a set of trestles. This temporary support system provides a clear roadway of approximately 30m diagonally, meeting the requirement of no interference with traffic movement. The total weight of the girder segment to be lifted is approximately 150MT, which is well within the available crane capacity of 200MT.

The analysis of these three models reveals that the design parameters—such as bending stress, deflection, and reaction—are significantly influenced by the choice of support arrangement, girder span, and temporary support system. *Model-1* serves as a baseline, while *Model-2* demonstrates the benefits of shifting the supports outward to improve load distribution. *Model-3*, with the use of temporary supports and a reduced span of 36m, offers significant advantages in terms of bending stresses, deflections, and overall stability during the construction phase. The proposed erection scheme for *Model-3*, utilizing temporary trestles and optimized segment lengths, ensures that the construction process remains efficient, safe, and in compliance with traffic requirements.

4) Construction Stages for Erection Scheme

The construction process for the superstructure follows a carefully planned sequence of stages, modeled in the software to simulate real-world behavior under various loading conditions. Each stage represents a specific phase in the construction of the girder system and its eventual transition to full operational status. The stages are as follows:

Stage-1: Initial Erection (Segments Simply Supported on Temporary Supports) In Stage-1, all the segments of both the tub girders are simply supported over the temporary trestles without any transverse connection. This stage simulates the scenario where the girder segments are lifted and lowered into place on the trestles. At this point, no transverse connections between the girders are made, and the individual segments are not yet linked together. Plan View (Fig 1): The segments are placed on temporary supports, and each segment is simply supported at the ends by the trestles.

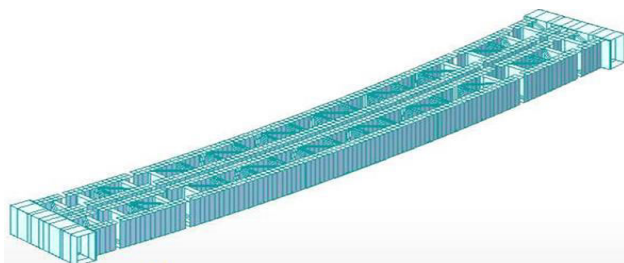


Fig 1: Stage 1- Plan (Segments simply supported on temporary supports)

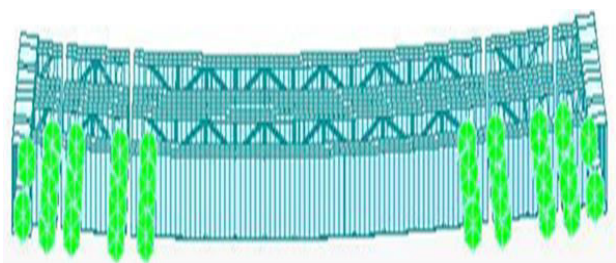


Fig 2: Stage 1- Elevation (Segments simply supported on temporary supports)

Elevation View (Fig 2): The elevation view shows the girder segments resting on the trestles without any transverse connections. This stage is purely for initial placement and alignment of the girder segments, ensuring they are positioned correctly before further assembly.

Stage-2: Transverse Connections and Longitudinal Splicing

In **Stage-2**, the segments of the outer tub girders are connected to the inner tub girder in the transverse direction to improve the structural stability of the system. At this point, all the segments of both tub girders are also connected together longitudinally through splicing. External cross frames and diaphragms are added to enhance the lateral stability, and high-strength friction grip (HSFG) bolts are used to splice the segments together longitudinally.

Plan View (Fig 3): The segments are shown with transverse connections and longitudinal splicing in place, ensuring a more rigid connection between the girders. The addition of cross frames and diaphragms at this stage improves overall stability.

This stage is crucial for ensuring that the girder segments are properly secured together to maintain stability under the upcoming loading conditions.

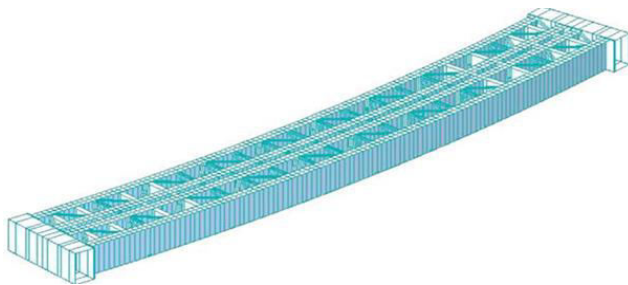


Fig 3: Stage 2 – Plan (Segments connected transversely and spliced longitudinally)

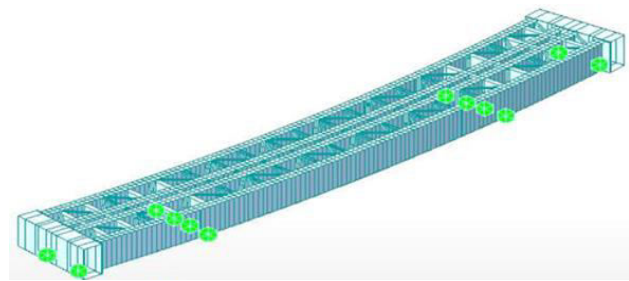


Fig 4: Stage 3 – Plan (Pouring of concrete)

Stage-3: Activation of Concrete Pouring and Removal of Temporary Supports

In **Stage-3**, the temporary supports (other than those supporting the 36m long segments) are removed to transform the structure into a continuous three-span unit. The removal of the temporary supports allows for proper bending behavior of the girders during the pouring of the concrete. The jacks that were previously used to support the girders are deactivated. Additionally, the load of the wet concrete is activated, representing the beginning of the concrete pouring process.

Plan View (Fig 4): The temporary supports are removed, and the structure is now a continuous unit. The concrete pouring is assumed to occur in a single go, given the girder's span of 60m and width of 8.50m. This stage signifies a major transition in the construction process, where the girder segments are integrated with the poured concrete to form a continuous structure.

Stage-4: Removal of All Temporary Supports and Activation of Composite Properties

In **Stage-4**, all the temporary supports are fully removed, and the structure is now supported by permanent bearings. The composite properties of the girder are activated, reflecting the full-strength deck concrete that has been curing for 28 days since the last stage. During this stage, the superimposed dead loads are activated, accounting for the weight of the deck slab, parapet walls, and other fixed components. **Plan View (Fig 6):** The permanent bearings are now active, and the structure is supported as a complete unit, ready for its final functional phase. All superimposed dead loads (SIDL) are accounted for, including the parapet walls, plinths, and other components necessary for the metro's operation.

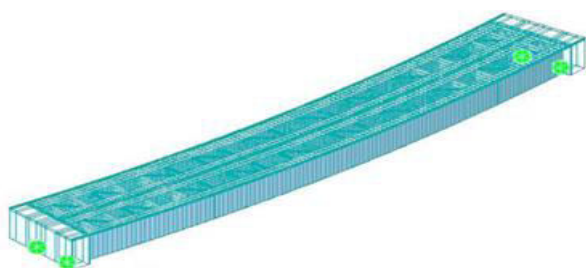


Fig 5: Stage 5 – Plan (SIDL stage)

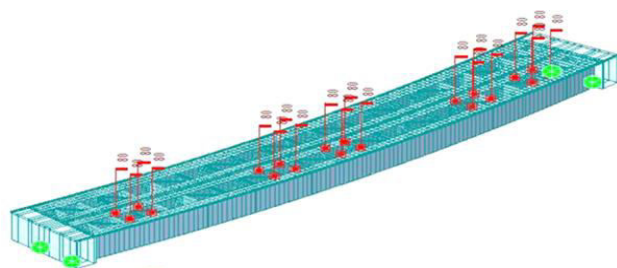


Fig 6: Stage 6 – Plan (Service stage)

This stage represents the completion of the main structural components and prepares the system for the final operational loads.

Stage-5: Activation of Metro Loads for Operational Stage

In Stage-5, the final load of the metro train is activated. This stage simulates the full operational condition, where the structure supports the dynamic loads of the running metro trains. The structure is now fully operational, with all components in place, including the track systems, parapet walls, and operational setups. Plan View (6): This plan view represents the structure at its final service stage, where the metro load is activated, and the structure is ready for use. The load from the running metro trains is applied to simulate the dynamic forces the bridge will experience during normal operation. This stage marks the transition from construction to service, where the structure must meet all operational requirements and continue to perform effectively under traffic loads.

Each construction stage is a critical step in ensuring the safe and efficient assembly of the girder structure. The stages transition from initial placement and temporary support systems to full operational capability, with each phase carefully designed to enhance the overall stability, safety, and performance of the structure. Through careful planning of the erection scheme, including the use of temporary supports, cross frames, and diaphragms, the structure can be efficiently and safely completed to withstand the operational loads once the metro system is in service.

IV. EXPERIMENTAL RESULTS

The analysis for live load, superimposed dead load, and dead load is summarized in **Table 1** for **Model-1**, showing the reactions at the four bearings.

Table 1: Reaction Summary at service stage (in MT) Model-1

Sr. No.	Loading	P1 (Inner bearing)	P1 (Outer bearing)	P2 (Inner bearing)	P2 (Outer bearing)
1	Dead Load	128.9	468.3	128.8	468.4
2	SIDL	44.8	186.8	44.8	186.9
3	Live Load	-27	128	-27	128
4	Total	146.7	783.1	146.6	783.3

1) Reaction Summary at Service Stage

In *Model-1*, the reactions for the **dead load**, **SIDL**, and **live load** (when the metro train runs) are evaluated, as shown in Table 1. The total reaction for dead load and superimposed dead load is about **146.7 MT** at the inner bearing (P1) and **783.1 MT** at the outer bearing (P2). The structure is stable at this stage with no uplift reactions at the inner bearings. The uplift due to centrifugal forces and seismic forces is considered in the next model.

2) Reaction Summary for Model-2 (Shifting Supports for Improved Stability)

In *Model-2*, the supports are shifted towards the outer side of the curvature by 0.4m to improve stability under centrifugal and seismic forces. This model helps increase the downward reaction at the inner bearings. As seen in **Table 4.2**, the reactions in *Model-2* show that the reactions on the inner bearings are significantly increased:

Table 2: Reaction Summary at service stage (in MT) Model-2

Loading	P1 (Inner bearing)	P1 (Outer bearing)	P2 (Inner bearing)	P2 (Outer bearing)
Dead Load	187.2	410	187.1	410.1
SIDL	67.4	164.2	67.4	164.3
Live Load	-19.4	117.9	-19.4	117.9
Total	235.2	692.1	235.1	692.3

Model-2: Inner bearing reactions (P1) are **235.2 MT** and **235.1 MT** for the two supports, compared to **146.7 MT** in **Model-1**. This represents a **60% increase** in the reactions on the inner bearings, improving the overall stability of the structure.

3) Summary on Stability

The results from the reactions at various stages clearly show that the structure is stable during all stages of construction and service. In *Model-1*, the structure is stable, but the reactions on the inner bearings are insufficient to handle uplift forces due to centrifugal and seismic forces. *Model-2* improves the stability of the structure by shifting the supports outward, which increases the downward reactions at the inner bearings by approximately 60% compared to *Model-1*. This enhances the stability of the structure during service under various loading conditions.

Thus, the stability of the structure is significantly improved by adjusting the support positions, making *Model-2* a more robust solution for dealing with the challenges posed by centrifugal and seismic forces.

4) Deflection Analysis of the Metro Bridge Structure

Deflection analysis is crucial for understanding how the bridge will behave under various loads, ensuring that the deflections remain within permissible limits. Excessive deflection can lead to discomfort for passengers, particularly in a metro bridge where smooth and stable movement is essential. In this section, deflections at different stages and due to various loadings are evaluated. A deflection camber is proposed to ensure the bridge remains within the allowable deflection limits.

Deflection Analysis at Various Stages

1) Deflection Due to Self-Weight of the Composite Section (Dead Load)

Fig 7 shows the deflection of the bridge due to the dead load (self-weight of the composite section). This deflection is calculated under the assumption that the structure is fully composite, i.e., the concrete deck and the girder work together as a single unit.

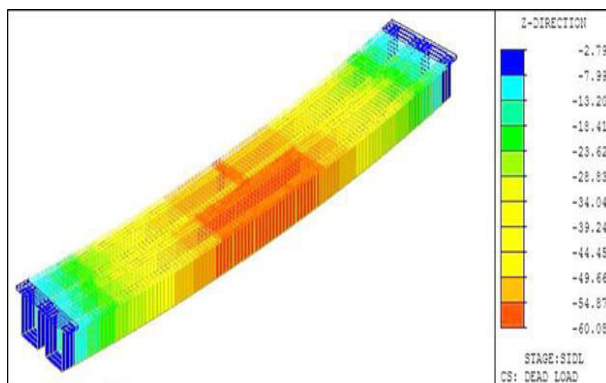


Fig 7: Deflection due to Dead Load (mm)

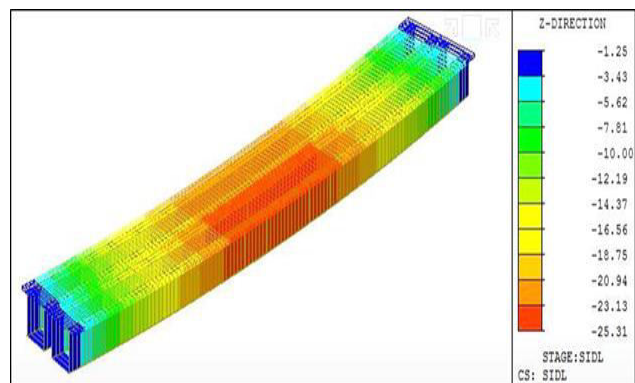


Fig 8: Deflection due to Super Imposed Dead Load (mm)

2) Deflection Due to Superimposed Dead Load (SIDL)

Fig 8 shows the deflection due to superimposed dead loads (SIDL) over the span. These are the loads added to the structure apart from the self-weight, such as parapet walls, railings, etc.

3) Deflection Due to Live Load (Metro Train Running Over the Bridge)

Fig 9 shows the deflection due to the live load (in this case, the metro train running over both tracks of the bridge). Live load deflections are critical in understanding how the structure behaves under dynamic loading from train traffic.

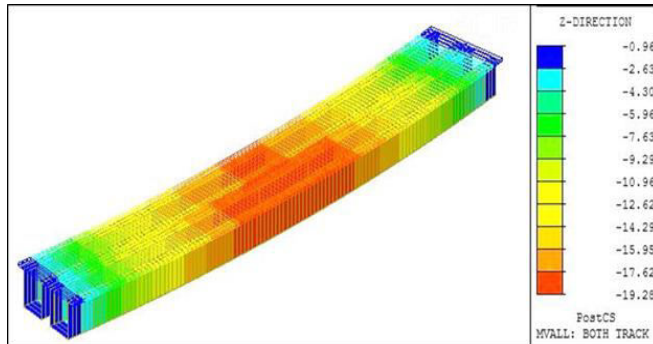


Fig 9: Deflection due to Live load (mm)

Table 3: Deflection at service stage (in mm)

Loading	For Inner Girder	For Outer Girder
Dead Load	54.91	60.08
SIDL	23.11	25.31
Live Load	17.82	19.28
Total (DL+1.068*SIDL+1.15*LL)	100	110

4) Deflection Summary at Service Stage

The overall deflection at service stage (considering dead load, superimposed dead load, and live load) is summarized in Table 3. The total deflection, considering all loads, is calculated as: Inner Girder: 100 mm and Outer Girder: 110 mm.

5) Comparison with Permissible Deflection Limits

According to Clause 4.17 of the IRS Steel Bridge Code, the deflection-to-length ratio should not exceed 1/600 for the serviceability limit state (SLS). For a girder of length 60,000 mm (60 m), the permissible deflection would be 100 mm. However, the calculated deflections at service stage are: Inner Girder: 100 mm and Outer Girder: 110 mm. Since the deflection of the outer girder (110 mm) exceeds the permissible limit of 100 mm, the structure is not within the allowable deflection limits.

Proposed Solution:

Camber to Avoid Excessive Deflection

To address the issue of excessive deflection, particularly sagging (which could cause discomfort to passengers), a camber of 100 mm is proposed for the structure. This camber will compensate for the deflection caused by the applied loads, ensuring that the deflections are within the permissible limits at service stage.

The structure will experience deflections that exceed the permissible limits according to the IRS code. To counteract this, a camber of 100 mm is proposed, which will help ensure that the bridge stays within the acceptable deflection range during its service life.

V. CONCLUSIONS

Based on the construction stage analysis following conclusions are drawn

1. The constructability of the complex superstructure, with a plan curvature radius of 225m and a span length of 60m, can be achieved without disrupting traffic, provided that the construction stages are carefully planned.
2. The design of the superstructure is primarily governed by deflection limits rather than stress limits, ensuring the structure maintains the required stability and performance during construction and operation.
3. The stress in the compression flange, due to the self-weight and deck concrete of the girder-only section, is reduced by 75% by reducing the span length to 36m from the original 60m. This modification enhances the safety of the structure during the deck casting process.
4. The stability of the inner bearing is improved by 60% by shifting the supports 0.4m towards the outer side of the curvature, away from the centerline, which helps enhance the load distribution and support during construction.
5. Additional stability at the supports on the inner side of the curvature is achieved when the sequence of pouring is carried out from the supports towards midspan, and from the inner side of the curvature towards the outer side.
6. The torsional effects at the supports, caused by the sharp curvature, are mitigated more effectively with the use of a box diaphragm rather than an I-section diaphragm, ensuring better resistance to twisting forces.
7. The load distribution between the twin-tub girders can be optimized by providing external bracings and diaphragms, which help distribute loads more evenly and improve the overall structural integrity.

Through the application of fundamental engineering principles and careful attention to the construction stages, the design parameters of the superstructure have been significantly improved. The structure has been made safer and more

stable during both the construction phase and the service stage. By optimizing key factors such as span length, bearing support placement, pouring sequence, torsional resistance, and load distribution, the structure achieves better performance under seismic and dynamic loading conditions. These improvements not only enhance the safety and stability of the superstructure during construction but also ensure its longevity and functionality once in service.

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