Behaviour of Curved Steel Tub Girder for Elevated Metro Rail

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Abstract — **The aim is to provide an effective solution for construction of superstructure for an obligatory span of 60m length with plan radius of 225m. The alignment of metro rail along a certain existing road changes its direction on either left or right at major junctions along crossing road. In such cases obligatory spans with sharp curvatures are inevitable. Usually a precast segmental superstructure using launcher is proposed for metro with standard span up to 35m, but in case of major junctions, span length can exceed up to 60m. In such cases, choice of superstructure is a challenging task considering sharp curvature, span length and feasibility of execution at a busy junction. Based on this, a typical obligatory span curved in plan is selected for further studies. The study is carried out for a curved span of 60m with radius of 225m; width of viaduct 8.5m for a two track metro line and tub girder composite superstructure using grillage analysis with construction stages in MIDAS software. The behaviour of superstructure during service stage using parameters such as Bending Stress and deflection is also present here.**

Index Terms — **An obligatory span, Bending Stress and deflection, Curved span, Trestle location.**

I. INTRODUCTION

Rapid transit or mass rapid transit, also known as heavy rail, metro, subway, tube or underground, is a type of high capacity public transport generally found in urban areas. Unlike buses rapid transit systems are electric railways that operate on an exclusive right of way, which cannot be accessed by pedestrians or other vehicles of any sort, and which is often grade separated in tunnels or on elevated railways.

Metropolis cities in India have witnessed enormous industrial growth in 21st century. Rapid urbanization in recent past has put travel infrastructure to stress leading to longer travel time and increased air pollution. Traffic in these cities is rising at an alarming rate. Being densely populated and having narrow roads, the transportation needs cannot be met by road based system and additional flyovers. In light of this, a high capacity public transport system needs to be proposed. At present there are a couple of alternatives such as conventional railway and BRTS, but it requires dedicated corridor, which is practically not possible in metro cities. Therefore rapid transit system like Metro is the only solution.

Metro rail is an electric railway that operates on an exclusive right of way which is grade separated in either tunnel or on elevated railway. Elevated railway is a cheaper, easier and feasible way to build an exclusive right of way due to considerations such as expensive tunneling and space constraint at road level in densely populated urban areas. Except for the construction period, it occupies little space at median, making it more feasible than any other alternative available.

In the late 1960s, a research group called the Consortium of University Research Teams (CURT) was formed to study the behaviour of horizontally curved bridges in USA. Analytical and experimental research was performed throughout the 1970s as part of the CURT project. The guide specifications were disjointed and difficult to follow. Therefore it was never been adopted as an integral part of the AASHTO Standard Specification for Highway Bridges (AASHTO 1992). In 1992 the Federal Highway Administration, along with 13 states, began a project to study the behaviour of horizontally curved steel bridges. The project is referred to as the Curved Steel Bridge Research Project and involves studying the behaviour of curved members through theoretical, analytical, and experimental research. In this chapter the work of researchers related to curved steel bridges is discussed.

Charles W. Roeder *et al* [1] performed experiments on a specially designed rig that applies a constant compressive load, a constant shear force, and a cyclic rotation. Sixteen tests were performed on pot bearings, four on spherical bearings, and two on disk bearings. The test results are summarized, and general observations on the bearing behavior and design considerations are noted in this paper. Each bearing type has advantages and potential problems. Pot bearings are able to support large compressive loads, but their elastomer might leak and their sealing rings might suffer wear or damage. Disk bearings are susceptible to uplift during rotation, which may limit their use in bearings with polytetrafluoroethylene (PTFE) sliding surfaces. Spherical bearings are able to sustain large rotations but require proper clearances, very smooth and accurate machining.

Further, James S. Davidson and Chai H. Yoo [2] analysed detailed finite-element models representing a curved three-girder test frame, which is planned under the Curved Steel Bridge Research Project experimental phase, are used to evaluate the effects of curvature on the bending strength of curved I-girders. Linear-elastic static, buckling, and combined material and geometric nonlinear analyses are conducted using models that represent the test frame and component test specimens that will be inserted into it. The results are compared to various predictor equations developed from analytical work by the writers and to related work by other researchers, including Japanese research that is not readily available in the United States. It is demonstrated that the predictor equations developed by the writers are accurate in representing the behavior of the system. Limitations and needed improvements are described as well.

Similarly, Zhanfei Fan and Todd A. Helwig [3] performed analytical study on distortional loads and brace forces in steel box girders. Typical torsional loads on curved box girders are discussed, and the distortional components of these applied torsional loads are studied. The distortional components from different torsional loads on trapezoidal box girders are identified and used to derive expressions for the brace forces in the internal cross frames for quasi-closed box girders. The results from the approximate equations for brace forces due to cross-sectional distortion are verified by three-dimensional finite-element analyses.

Nam-Hoi Park *et al* [4] performed extensive parametric studies using the box beam elements and the design charts for the maximum spacing of the intermediate diaphragms are presented. Bridges taken into account are single-span, two-span, and three-span continuous horizontally curved box girder bridges. The design parameters considered are the central angle, the number of spans, the span length, the aspect ratio of the box section, the spacing of the intermediate diaphragms, and the desired ratio of the distortional warping normal stress to the bending normal stress. Unlike the current practice where the ratio of the distortional warping normal stress to the bending stress is fixed to a specific value, the proposed design charts facilitate the determination of the maximum spacing of intermediate diaphragm for various desired stress ratios.

Magdy Samaan *et al* [5] performed an extensive parametric study, using a finite element method, in which the structural responses of 240 two-equal span continuous curved box-girder bridges of various geometries were investigated. The parameters considered in this study included span-to-radius of curvature ratio, span length, number of lanes, number of boxes, web slope, number of bracings, and truck loading type. Based on the data generated from this study, empirical formulas for load distribution factors for maximum longitudinal flexural stresses and maximum deflection due to dead load as well as AASHTO live loading were deduced.

Further, D. R. Popp and E. B. Williamson [6] observed a number of construction problems in steel trapezoidal box-girder bridges in recent years. Failures have ranged from the buckling of bracing members to the complete collapse of a pedestrian bridge during construction in Marcy, New York. These bridges met current design criteria, implying that constructability may be inadequately addressed in the specifications. Software has been developed to investigate the overall behavior of steel trapezoidal box-girder bridges during the construction phase. Unlike currently available design software, the new software explicitly accounts for partially-composite behavior between the girder and the deck. In addition, the developed program utilizes a finite-element-based procedure in which the exact geometry of the cross-section is discretized for the purposes of analysis. Previous research has demonstrated that such an approach is required to accurately predict the response of trapezoidal box-girder bridges under construction loads. Results for the current study have shown that lateral torsional buckling of the entire girder, which is not currently a limit state for design, must be considered. In this paper, the developed software is used to investigate the failure of the bridge in Marcy. Computed results are shown to agree with observations from the forensic investigation.

Kyungsik Kim and Chai H. Yoo [7] performed analytical study on interaction of top Lateral and internal bracing systems in tub girders. A lateral bracing system is installed at the top flange level of a steel tub girder to form a quasi-closed box, thereby increasing torsional resistance and retaining the original box shape. A single diagonal SD type or crossed diagonal type is typically adopted along with intermediate internal bracing systems also called internal cross frames that are placed at either even or odd numbered spacing. A significant coupling action between SD bracing members and internal cross-frame members has been found to occur in the case of a SD lateral bracing system and internal cross frames placed at odd numbered panel spacing. The member forces developed in lateral bracing are 25–30% higher than those in structures with internal cross frames installed at even numbered panel spacing. Matrix equations were formulated to compute the brace forces developed in both SD lateral bracing and K-shaped internal cross-frame members subjected to bending action. Member forces computed from the new equations were compared with those from three-dimensional finite-element analysis, and an excellent agreement between the two solutions was found.

Further, B. J. Bell and D. G. Linzell [8] prepared a model of a three-span continuous portion of a bridge in SAP 2000, which was the subject of several analyses exploring the effects erection sequencing, implementation of upper lateral bracing, and use of temporary supports had on the final deformed shape of the curved superstructure. Findings indicated that using paired girder erection produced smaller radial and vertical deformations than single girder techniques for this structure, and that the use of lateral bracing between the fascia and adjacent interior girders and the placement of temporary shoring towers at span quarter points are both effective means of further reducing levels of deflection.

Chai H. Yoo, *et al* [9] derived the interaction equations for predicting the nominal bending strength of horizontally curved I-girders of compact-flange sections subjected to vertical moment and torsion. Horizontally curved I-girders are subjected to combined vertical bending and torsion under gravity loads alone. The torsional behavior of open I-shaped girders is commonly and conveniently transformed to self-equilibrating lateral bending of flanges. The interaction of vertical bending and lateral flange bending reduces the vertical moment–carrying capacity of the section.

A new classification of singly symmetric I-shaped sections between compact and non compact sections is introduced. It is designated as the compact-flange section, which has compact flanges and a non compact web. It is practical for this section to have a web that barely satisfies the requirement of a non compact web. In the flexural strength check of a non compact I-shaped section, the maximum flexural stress at the flange-web juncture is limited to the yield stress. When the flange stresses are evaluated based on the combined actions of vertical bending and lateral bending, the capacity of a non compact section is considerably limited. The flange-compact section can overcome this potentially severe limitation.

Kuppumanikandan A. [10] did a parametric study on behaviour of curved box girder bridges by using finite element method. The parameters considered to present the behaviour of Single Cell Box Girder, Double Cell Box Girder and Triple Cell Box Girder bridges are radius of curvature, span length and span length to the radius of curvature ratio. These parameters are used to evaluate the responses of box girder bridges namely, longitudinal stresses at the top and bottom, shear, torsion, moment, deflection and fundamental frequency of three types of box girder bridges.

The parametric study on behaviour of box girder bridges showed that, as curvature decreases, responses such as longitudinal stresses at the top and bottom, shear, torsion, moment and deflection decreases for three types of box girder bridges and it shows not much variation for fundamental frequency of three types of box girder bridges due to the constant span length. It is observed that as the span length increases, longitudinal stresses at the top and bottom, shear, torsion, moment and deflection increases for three types of box girder bridges. As the span length increases, fundamental frequency decreases for three types of box girder bridges. Also, it is noted that as the span length to the radius of curvature ratio increases responses parameter longitudinal stresses at the top and bottom, shear, torsion, moment and deflection are increases for three types of box girder bridges. As the span length to the radius of curvature ratio increases fundamental frequency decreases for three types of box girder bridges.

The present work focuses on construction stage analysis of a tub girder superstructure. A grillage model is prepared in Midas civil based on the plan geometry available of a metro alignment. The analysis addresses stability of superstructure during erection sequence, pouring of deck concrete and at service stage and emphasize on passenger comfort by limiting deflection of superstructure to a certain degree specified by the metro authorities.

The aim of this study is to provide an effective solution for construction of superstructure for an obligatory span of 60m length with plan radius of 225m. The alignment of metro rail along a certain existing road changes its direction on either left or right at major junctions along crossing road. In such cases obligatory spans with sharp curvatures are inevitable. Usually a precast segmental superstructure using launcher is proposed for metro with standard span up to 35m, but in case of major junctions, span length can exceed up to 60m. In such cases, choice of superstructure is a challenging task considering sharp curvature, span length and feasibility of execution at a busy junction. Based on this, a typical obligatory span curved in plan is selected for further studies.

The study will be carried out for a curved span of 60m with radius of 225m; width of viaduct 8.5m for a two track metro line & tub girder composite superstructure using grillage analysis with construction stages in MIDAS software.

1.1 Geometry of the alignment

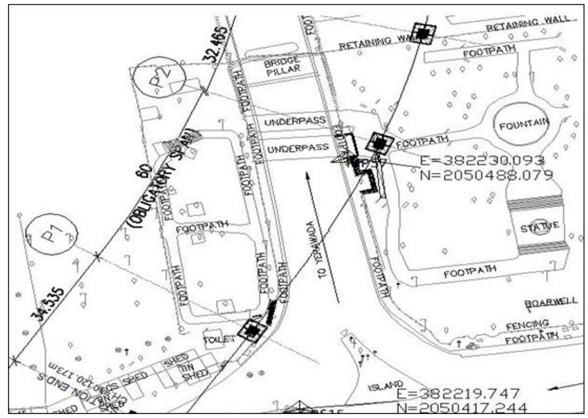

Fig 1: Alignment of the metro rail line at the junction

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.

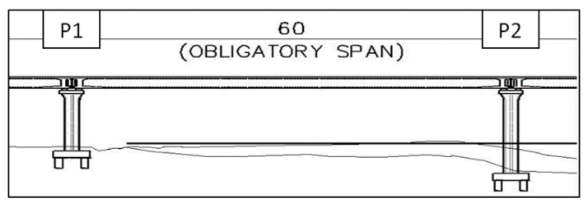

Fig 2: Elevation of obligatory span

1.2 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.

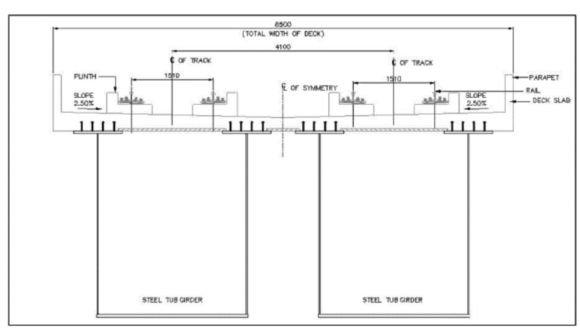

Fig 3: Cross section of the metro bridge

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 up to 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.

Models for the analysis

In view of the design optimization, three different models are prepared and analysed for various loadings and then compared on the basis of design parameters i.e bending stress, deflection and reaction. All the aspects as discussed prior in this chapter are same for these models except for some changes as discussed below.

Model-1, a single support is provided under each girder on the centreline of the girder.

Model-2, Positions of the supports shifted towards outer side of the curvature by 0.4m and the girder length of 60m is considered to be erected on bearings without intermediate temporary supports. In this case the bending stresses for self weight of girder and wet concrete are worked out with girder only section properties.

Model-3, Positions of the supports shifted towards outer side of the curvature by 0.4m and the girder length of 60m is considered to be erected with intermediate temporary supports by restricting maximum span up to 36m and the temporary supports are removed only after the girder section becomes composite i.e. after 28 days of casting of deck slab. In this case the bending stresses for self-weight of girder and wet concrete are worked out with girder only section properties only for the bending moment at construction stage with maximum span of 36m. The bending stresses for Dead Load, SIDL and Live Load for 60m span are worked out with composite section properties.

For model-3, construction stages depend on the erection scheme and the pouring sequence of the deck concrete. While deciding the erection scheme couple of aspects needs to be taken into account such as required clear roadway for the traffic and maximum crane capacity available to lift the girder segment.

Considering the above aspects, it is decided to divide the 60m length of the girder into five equal segments of length 12m each. Middle three segments will constitute a girder length of 36m. This length of girder can be supported temporarily over a set of trestles making a clear roadway of around 30m diagonally, thus satisfying the requirement of no interference to the traffic. Also, the weight of the girder to be lifted works out to be around 150MT which is well within the available crane capacity of 200MT.

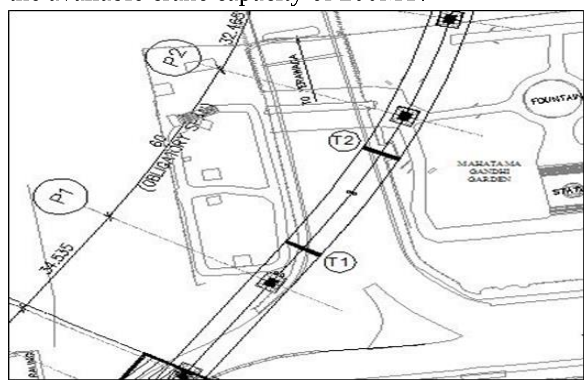

Fig 4: Trestle locations for proposed erection scheme

Trestle locations are highlighted in the above diagram. Trestle 'T1' and Trestle 'T2' are placed at 12m and 36m from the pier P1 respectively. It can be seen that there is enough clear roadway for the traffic movement. Further, end segments of 12m length are divided into two parts i.e. 3.5m length segment including box diaphragm (As the width of the box diaphragm is 8.50m, length of the segment needs to be restricted to 3.5m to facilitate transportation of the segment to the site.) and 8.5m length segment. To erect these segments, couple more trestles needs to be provided between pier and trestle T1/T2.

Based on this erection scheme, construction stages are prepared in the software.

Stage-1: In this construction stage all the segments of both the tub girders are simply supported over the temporary supports without any transverse connection. This stage represents the case where the segments will be lifted and lowered on trestles.

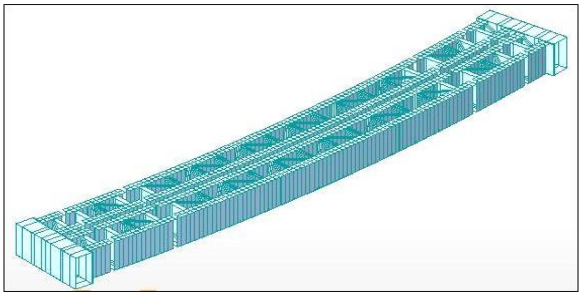

Fig 5: Stage 1- Plan (Segments simply supported on temporary supports)

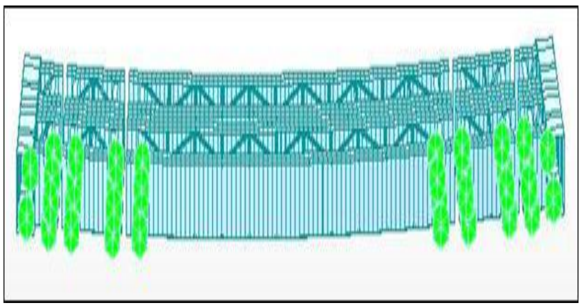

Fig 6: Stage 1- Elevation (Segments simply supported on temporary supports)

Stage-2: In this construction stage all the segments of outer tub girders are connected to inner tub girder in the transverse direction to ensure more stability and supports are same. This stage represents the case where the segments will be connected by external cross frames and diaphragms. Also all the segments of both the tub girders are connected together in longitudinal direction. This stage represents the case where the segments will be spliced together with HSFG bolts.

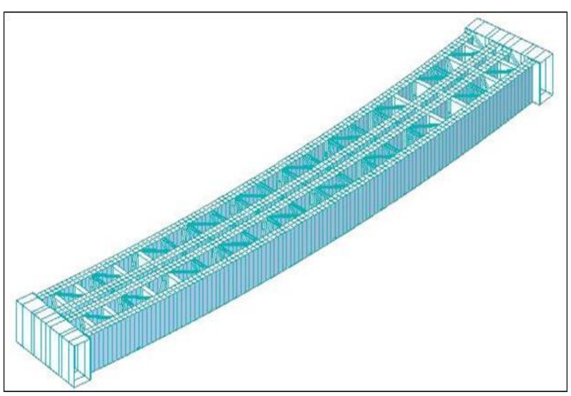

Fig 7: Stage 2 – Plan (Segments connected transversely and spliced longitudinally)

Stage-3: In this construction stage, temporary supports other than those supporting 36m long segments are deactivated to make the a three span continuous unit. These supports are removed to ensure proper bending behavior in the structure during pouring of concrete. In actual, the jacks used to support the girders temporarily, will be removed.

Also, wet concrete load is activated representing the pouring sequence. As the span is 60m long and 8.50m wide, concreting is assumed to be completed in a single go.

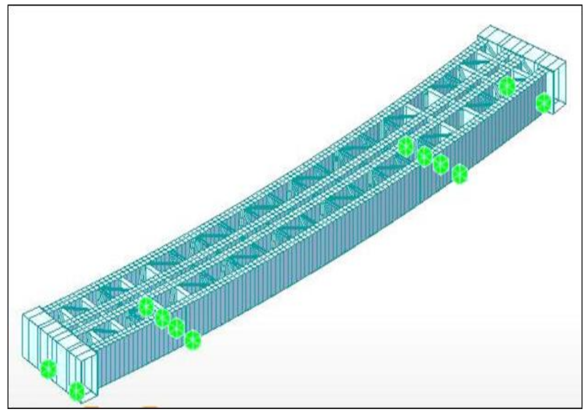

Fig 8: Stage 3 – Plan (Pouring of concrete)

Stage-4: In this stage all the temporary supports are removed, so now the structure is supported on permanent bearings and the composite property of the girder is activated considering the deck concrete has achieved full strength after 28 days of previous stage. Super imposed dead loads are activated. This stage represents the stage where parapet wall, plinth for the rails and all the necessary arrangement for the operation of the metro is completed.

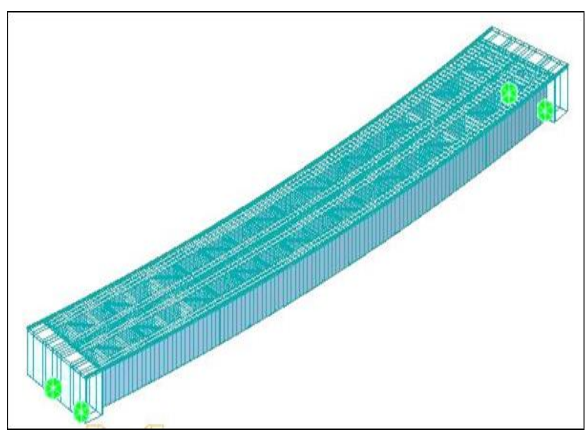

Fig 9: Stage 5 – Plan (SIDL stage)

Stage-6: In this stage metro load is activated. This stage represents the metro train running over the structure.

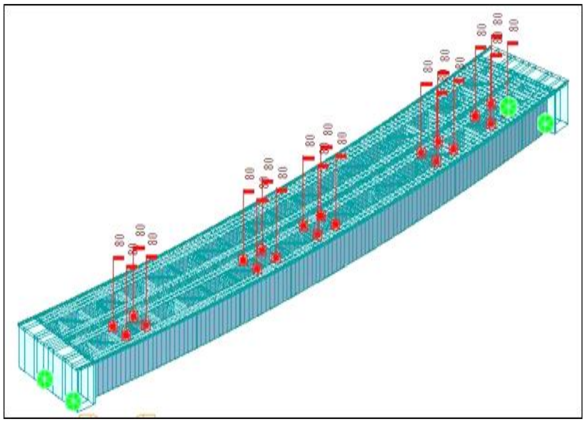

Fig 10: Stage 6 – Plan (Service stage)

II. RESULTS AND DISCUSSION

In this study, design parameters like reactions, deflection, flexural stresses are studied. A parametric comparison is done and conclusions are drawn.

A. Stability Analysis

Stage-1 Reactions are checked for the segments launched in position case.
It can be seen that there is no uplift reaction at any support on the inner side of the curvature. It is concluded that for a plan radius of 225m and temporary support spacing of 2.3m, maximum girder length of 36m is safe against overturning.

Stage-2 In this stage girders are connected to each other making it more stable than the case in stage-1.

Stage-3 Reactions are checked for pouring of concrete case. At this stage structure is more susceptible to uplift due to load from wet concrete on outer side of curvature, therefore sequencing of concrete pouring is of utmost importance. Considering this it is decided to pour deck concrete from the pier locations towards the midspan, also from inner side of the curvature towards the outer side of the curvature. This will ensure that there will be extra downward load on inner supports while pouring concrete at outer side of the curvature.
It can be seen that there is no uplift reaction at any support on the inner side of the curvature. It is concluded that for a plan radius of 225m temporary support arrangement is safe against overturning.

Stage-5 Reactions are checked for service stage. After the concrete has achieved full strength after 28 days of pouring of concrete, it is proposed to remove the remaining temporary supports.
At service stage the uplift at inner bearing needs to be checked for the metro train running on the outer track case.

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

The resultant reaction due to dead load, superimposed dead loads and uplift due to metro train on the outer track is checked. It works out to be 146 MT downward; hence it is concluded that the structure is stable during service stage.
Considering the uplift due to centrifugal forces and seismic forces, inner bearings needs to have sufficient downward reaction and for that purpose reactions on inner bearings are improved by shifting supports towards outer side of the curvature from centreline in Model-3.

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

Sr. No.	Loading	P1 (Inner bearing)	P1 (Outer bearing)	P2 (Inner bearing)	P2 (Outer bearing)
1	Dead Load	187.2	410	187.1	410.1
2	SIDL	67.4	164.2	67.4	164.3
3	Live Load	-19.4	117.9	-19.4	117.9
4	Total	235.2	692.1	235.1	692.3

From the results it can be concluded that the reactions on the inner bearings in Model-3 are increased by about 60% of the reactions of Model-1. Hence the stability of the structure is improved for the case of centrifugal and seismic forces.

B. Deflection Analysis

1) Deflection due to self-weight of the composite section.

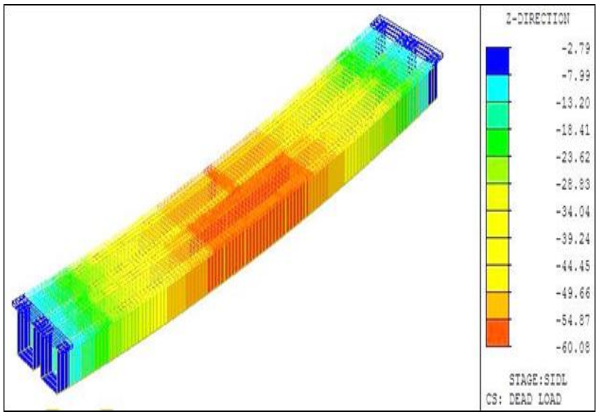

Fig 11: Deflection due to Dead Load (in mm)

2) Deflection due to super imposed dead loads over the span.

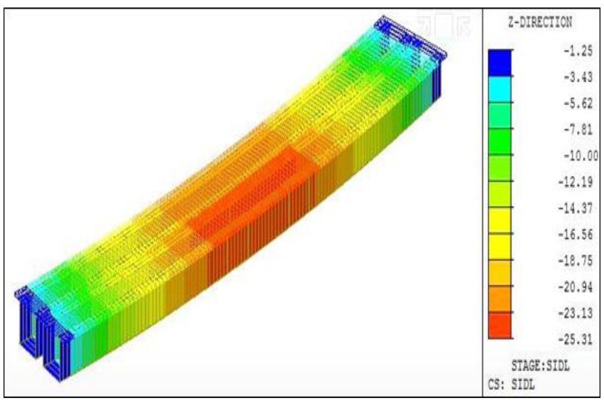

Fig 12: Deflection due to Super Imposed Dead Load (in mm)

3) Deflection due to metro train both track loaded case.

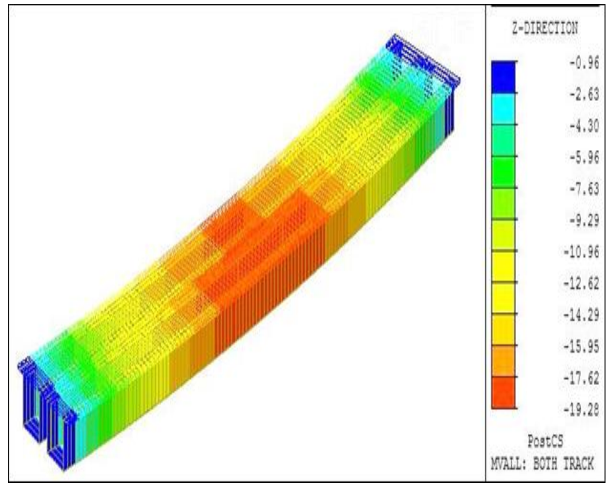

Fig 13: Deflection due to Live load (in mm)

Table 3: Deflection Summary at service stage (in mm)

Sr. No.	Loading	For Inner Girder	For Outer Girder
1	Dead Load	54.91	60.08
2	SIDL	23.11	25.31
3	Live Load	17.82	19.28
4	Total (DL+1.068*SIDL+1.15*LL)	100	110

As per clause no. 4.17 IRS steel bridge code, the ratio of deflection to length of the girder shall not exceed 1/600 (i.e. (60000)/600 = 100mm). For SLS combination the deflection works out to be (60.08+1.068*25.31+1.15*19.28=110mm) It can be seen that the deflection exceeds the permissible limits. Hence to avoid the sagging effect in the superstructure, which will eventually cause discomfort to the passengers when the train passes over the span, camber of 100mm is proposed for the structure.

C. Flexural Analysis

Flexural stresses are worked out for Model-2 and Model-3 and compared see the extent of design optimization.

Table 4: Bending Stress Summary (in MPa) for Model-2

Sr. No.	Loading	At Mid span	
		Girder Top	Girder Bottom
1	Self-weight for 60m span	61.39	- 45.20

2	Wet concrete for 60m span	40.63	- 29.62
4	SIDL for 60m span	18.25	- 26.73
5	Live Load	15.17	- 21.64
6	CDA factor	1.2	1.2
7	Live Load with Impact	18.20	- 25.97
8	Temperature	5.88	1.9
9	Total (Permissible 195 MPa)	144.35	125.26

Note – '- ve' sign denotes tensile stress

In above table bending stresses for self-weight of girder and wet concrete loads are worked out using girder section alone properties and for other loadings, stresses are worked out using composite section properties.

Table 5: Bending Stress Summary (in MPa) for Model-3

Sr. No.	Loading	At Midspan	
		Girder Top	Girder Bottom
1	Self weight for 36m span	17.80	- 13.22
2	Wet concrete for 36m span	5.88	- 4.19
3	Dead Load for 60m span	57.36	- 69.71
4	SIDL for 60m span	19.79	- 27.71
5	Live Load	16.06	- 22.02
6	CDA factor	1.2	1.2
7	Live Load with Impact	19.27	- 26.42
8	Temperature	5.88	1.9
9	Total (Permissible 195 MPa)	126	140

Note – '- ve' sign denotes tensile stress

In above table stresses for loadings at Sr. No. 1 & 2 are worked out using girder section alone properties and for other loadings, stresses are worked out using composite section properties.

Following are the observations on the comparative study of the analysis results for model-2 and model-3 using design parameter bending stress.

- At the construction stage of deck casting, the stress in compression flange for model-2 is more than 100 MPa, which makes the girder highly susceptible to the lateral torsional buckling. This will require larger sections for transverse members to effectively restrain the compression flange.
- By introducing temporary supports in model 3, the effective span length is reduced drastically reducing stress in compression flange by almost 75%. This will make sure the structure is safer during construction stage of deck casting.

Also from the above results it is observed that the stresses in girder are well within the permissible limits of 195 MPa. Hence it is concluded that for the section considered, the design is governed by the deflection criteria rather than stress limit criteria.

III. CONCLUSION

Based on the construction stage analysis following conclusions are drawn

1. Constructability of complex superstructure (Radius of plan curvature 225m and span length of 60m) can be achieved without interference to the traffic by proper planning of construction stages.
2. The design of the superstructure is governed by the deflection limit rather than stress limit.
3. Stress in compression flange due to self-weight and deck concrete for girder only section is reduced by 75% by reducing the span length to mere 36m instead of 60m, making the structure safer during deck casting.
4. Stability of inner bearing is improved by 60% by shifting supports by 0.4m towards the outer side of the curvature from the centreline.
5. More stability at supports at inner side of curvature is ensured when the sequence of pouring is done from the supports towards midspan and from inner side of the curvature towards outer side of the curvature.
6. Torsion at the supports due to sharp curvature is negotiated with greater effect with provision of box diaphragm rather than I section diaphragm.
7. The load distribution between twin tub girders can be improved by providing external bracings and diaphragms.

Hence it is observed that the design parameters are improved by application of basic principles to make the structure stable and safe during construction stages as well as in service stage.

REFERENCE

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or wrought iron bridges carrying rail, road or pedestrian traffic.”
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