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Design of Elevated Metro Bridge

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Abstract: *A metro system is a railway transport system in an urban area with a high capacity, frequency and the grade separation from other traffic. Metro System is used in cities, agglomerations, and metropolitan areas to transport large numbers of people. An elevated metro system is more preferred type of metro system due to ease of construction and also it makes urban areas more accessible without any construction difficulty. An elevated metro system has two major elements pier, piles cap, and box girder. The present study focuses on two major elements, pier, piles cap and box girder, of an elevated metro structural system.*

The performance assessment of selected designed pier showed that, the Force Based Design Method may not always guarantee the performance parameter required and in the present case the pier achieved the target requirement. In case of Direct Displacement Based Design Method, selected pier achieved the behaviour factors more than targeted Values. These conclusions can be considered only for the selected pier.

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.

In the future, India wants to extent its existing metro system. An elevated metro system high above the city is one of the possible concepts in this concept and moreover in whether there can be gained profit on the elevated metro structure by applying Ultra High-Performance Concrete or Fibre Reinforced Polymers instead of conventional concrete. The objective is to determine the dimensions and normative structural verifications of the elevated metro structure when this is made of conventional concrete, Ultra High-Performance Concrete or Fibre Reinforced Polymers and to compare these designs with each other.

The design study concerns the structural design and analysis of the elevated metro structure and results in three designs made of respectively conventional concrete, Ultra High-Performance Concrete (UHPC) and Fibre Reinforced Polymers (FRP). The literature and preliminary study give information about important aspects of elevated metro systems, UHPC and FRP and has as major objective to determine the height and span of the elevated railway for the application of an elevated metro system in India.

Ultra-High-Performance Concrete (UHPC) is a result of the search for a concrete with a higher strength. The strength classes of UHPC range between C90/105 and C200/230. The creation of UHPC is made possible by changing the design of the concrete mix by: improving the homogeneity and the microstructure, increasing the package density, adding steel fibres and reducing the water cement ratio. The material has a high durability and can result in more slender structures. The costs are however high compared with conventional concrete. Furthermore, the mix design of UHPC is complex and deserves special attention. It is assumed to be the best to utilize precast UHPC elements instead of in-situ UHPC. For the design of the elevated railway made from UHPC in the design study a thesis is used.

Fibre Reinforced Polymers (FRP) is a composite material. FRP consists of load-bearing fibres and a polymer resin matrix in which they are embedded. Whereas the fibres exercise the actual load-bearing function, the polymer matrix essentially has four functions:

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• Fixing the fibres in the desired geometrical arrangement
• Transferring the forces to the fibres
• Preventing buckling of the fibres under compression actions
• Protecting the fibres from humidity etc.

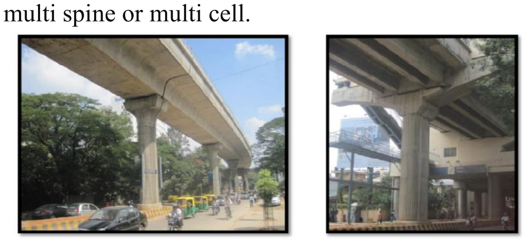
There is a wide range of FRP producible which has resulted in few standard composites and standard codes. FRP has a very high strength at low weight and by adding additives it can be mixed for many suitable applications. And as there is a wide range of FRP producible there are also many manufacturing processes. Points of interest are the possibility of delamination and the stiffness of the bridge. The cost for FRP is relatively high. Sandwich construction is commonly used with composites to increase structural efficiency, with the FRP forming the outer skins and bonded to a variety of core materials. For the design of the elevated railway made from FRP in the design study a thesis is used..

Keywords: Metro Bridge

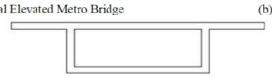
I. INTRODUCTION

A metro system is an electric passenger railway transport system in an urban area with a high capacity, frequency and the grade separation from other traffic. Metro System is used in cities, agglomerations, and metropolitan areas to transport large numbers of people at high frequency. The grade separation allows the metro to move freely, with fewer interruptions and at higher overall speeds. Metro systems are typically located in underground tunnels, elevated viaducts above street level or grade separated at ground level. An elevated metro structural system is more preferred one due to ease of construction and also it makes urban areas more accessible without any construction difficulty. An elevated metro structural system has the advantage that it is more economic than an underground metro system and the construction time is much shorter.

An elevated metro system has two major components pier and box girder. A typical elevated metro bridge model is shown in Figure 1. (a). Viaduct or box girder of a metro bridge requires pier to support each span of the bridge and station structures. Piers are constructed in various cross-sectional shapes like cylindrical, elliptical, square, rectangular and other forms. The piers considered for the present study are in rectangular cross section and it is located under station structure. Box girders are used extensively in the construction of an elevated metro rail bridge and the use of horizontally curved in plan box girder bridges in modern metro rail systems is quite suitable in resisting torsional and warping effects induced by curvatures. The torsional and warping rigidity of box girder is due to the closed section of box girder. The box section also possesses high bending stiffness and there is an efficient use of the complete cross section. Box girder cross sections may take the form of single cell, multi spine or multi cell.

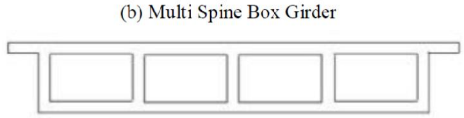


(a) Typical Elevated Metro Bridge (b) Typical Pier



(a) Single Cell Box Girder

(b) Multi Spine Box Girder



(c) Multi Cell Box Girder

1.2 Problem Description

Building underground metro systems is very expensive and takes a lot of time to realise. Besides, it is often a risky operation in urban areas. In areas with high land prices and dense land use, this option may however be the only economic route for mass transportation. The construction of ground level metro lines is the cheapest of the three options, as long as the land values are low. Since ground level metro lines create a physical barrier that hinders the flow of people and vehicles it is mostly used outside dense urban areas. Elevated railways are a cheap and easy way to build an exclusive metro line without digging expensive tunnels or creating physical barriers. Considering this from a practical and economical point of view, an elevated metro system is often the most suitable solution of the three options.

In some metropolises the infrastructure is elevated up to a large height above the city. In the India this concept can also be found, but often concerns a practical elevation of about 5 meters. This elevation allows car traffic to pass underneath. Due to the limited height, this is however often seen as a psychological barrier between two areas. By increasing the elevation as is applied in some metropolises this psychological barrier decreases. This makes the concept more attractive as alternative for the extension of the public transport. Moreover, as mentioned above an elevated metro system has the advantages that it costs less and takes less time to construct compared with an underground metro system and does not create a physical barrier. With a higher elevated metro system, it is thus possible to create an even more attractive alternative as it is also accepted more from a social point of view. This all makes this concept truly worth to take into consideration as option for mass transportation by metros.

In the future, India wants to extent its existing metro system. An elevated metro system high above the city is one of the possible concepts. The engineering office of India Public Works is interested in this concept and moreover in whether there can be gained profit on the elevated metro structure by applying Ultra High-Performance Concrete or Fiber Reinforced Polymers instead of conventional concrete. More specific, they would like to know if an elevated metro structure made of Ultra High-Performance Concrete or Fiber Reinforced Polymers results in different structural dimensions. Besides, the question is what the normative structural verifications are when these materials are applied to an elevated metro structure.

Notice that the title of this thesis contains the term “composite”. The term “composite” covers a wide range of material combinations. However, in this thesis “Fibre Reinforced Polymers” is meant with the term “composite”.

II. LITERATURE REVIEW AND REFERENCE PROJECTS

2.1 Design of Pier

Conventionally the pier of a metro bridge is designed using a force-based approach. Recent studies (Priestley et al., 2007) show that the force-based design may not necessarily guarantee the required target performances. The codes are now moving towards a performance-based design approach, which consider the design as per the target performances at the design stage. As the present study focus on the application of displacement-based approaches to pier design, a brief introduction of the two methods, force-based and displacement-based design is summarized in the following sections.

2.2 Force Based Design Method

Force Based Design Method (FBD) is the conventional method to design the metro bridge pier. In Force based design method, the fundamental time period of the structure is estimated from member elastic stiffnesses, which is estimated based on the assumed geometry of the section. The appropriate force reduction factor (R) corresponding to the assessed ductility capacity of the structural system and material is selected in the force-based design and applied to the base shear of the structure.

The design of a pier by force based seismic design method is carried out as per IS 1893: 2002 Code. The design procedure to find the base shear of the pier by FBD method is summarized below.

Step 1: The structural geometry of the pier is assumed.

Step 2: Member elastic stiffness are estimated based on member size.

Step 3: The fundamental period is calculated by: $T = 0.075\ h0.75$

Where h = Height of Building, in m

Step 4: Seismic Weight of the building (W) is estimated.

Step 5: The design horizontal seismic coefficient Ah for a structure determined by

$$A_h = \frac{Z\, I\, S_a}{2\, R\, g}$$

Where, Z = Zone factor
I = Importance factor
R = Response reduction factor,
Sa/g = Average response acceleration coefficient
Z, I, R and Sa/g are calculated as per IS 1893:2002 Code.
Step 6: The total design lateral force or design seismic base shear force (VB) along any principal direction is given by
VB = Ah W
Where Ah = Design Horizontal Seismic Coefficient and
W= Seismic Weight of the Building

2.3 Box Girder Bridges

In the past three decades, the finite element method of analysis has rapidly become popular and effective technique for the analysis of box girder bridges. So many researchers conducted studies on Box girder bridges by using finite element method. Khaled et al. (2001, 2002) have conducted detailed literature review on analysis of box girder bridges. Based on Khaled et al. (2001, 2002), the following literature review has been done and presented. Malcolm and Redwood (1970) and Moffatt and Dowling (1975) studied the shear lag phenomena in steel box-girder bridges.

Sisodiya et al. (1970) approximated the curvilinear boundaries of finite elements used to model the curved box-girder bridges by a series of straight boundaries using parallelogram elements. This approximation would require a large number of elements to achieve a satisfactory solution. Such an approach is impractical, especially for highly curved box bridges.

Komatsu and Nakai (1966, 1970) presented several studies on the free vibration and forced vibration of horizontally curved single, and twin box-girder bridges using the fundamental equation of motion along with Vlasov's thin-walled beam theory. Field tests on bridges excited either by a shaker or by a truck travelling at various speeds showed reasonable agreement between the theory and experimental results.

Chu and Pinjarkar (1971) proposed a finite element formulation of curved box-girder bridges, consisting of horizontal sector plates and vertical cylindrical shell elements. The method can be applied only to simply supported bridges without intermediate diaphragms.

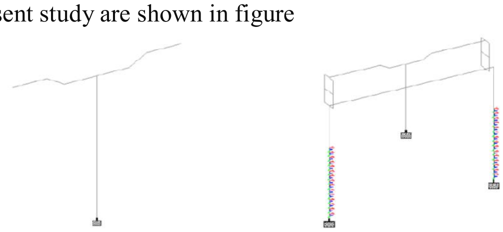
Chapman et al. (1971) carried out a finite element analysis on steel and concrete box-girder bridges to study the effect of intermediate diaphragms on the warping and distortional stresses.

Lim et al. (1971) proposed an element that has a beam-like-in-plane displacement field. The element is trapezoidal in shape, and hence, can be used to analyse right, skew, or curved box-girder bridges with constant depth and width.

William and Scordelis (1972) presented an elastic analysis of cellular structures of constant depth with arbitrary geometry in the plane using quadrilateral elements.

III. PERFORMANCE STUDY OF A PIER DESIGNED BY FBD AND DDBD

The geometry of pier considered for the present study is based on the design basis report of the Bangalore Metro Rail Corporation (BMRC) Limited. The piers considered for the analysis are located in the elevated metro station structure. The effective height of the considered piers is 13.8 m. The piers are located in Seismic Zone II, as per IS 1893 (Part 1): 2002. The modelling and seismic analysis is carried out using the finite element software STAAD Pro. The typical pier models considered for the present study are shown in figure



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The material property considered for the present pier analysis for concrete and reinforcement steel are given in Table

Properties of Concrete	
Compressive Strength of Concrete	60 N/mm^2
Density of Reinforced Concrete	24 kN/m^3
Elastic Modulus of Concrete	36000 N/mm^2
Poisson's Ratio	0.15
Thermal Expansion Coefficient	1.17 x 10^{-5} /^{0}C
Properties of Reinforcing Steel	
Yield Strength of Steel	500 N/mm^2
Young's Modulus of Steel	205,000 N/mm^2
Density of Steel	78.5 kN/m^3
Poisson's Ratio	0.30
Thermal Expansion Coefficient	1.2 x 10^{-5} /^{0}C

3.1 Design Load

The elementary design load considered for the analysis are Dead Loads (DL), Super Imposed Loads (SIDL), Imposed Loads (LL), Earthquake Loads (EQ), Wind Loads (WL), Derailment Load (DRL), Construction & Erection Loads (EL), Temperature Loads (OT) and Surcharge Loads (Traffic, building etc.) (SR). The approximate loads considered for the analysis are shown in Table. The total seismic weight of the pier is 17862 kN.

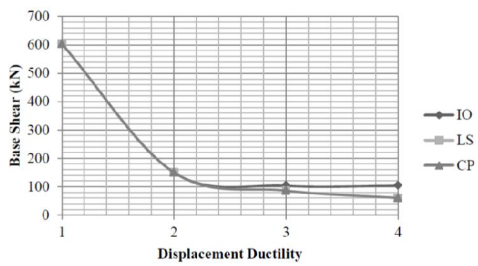
Load from Platform Level	**Load**	**Load from Track Level**	**Load**
Self Weight	120 kN	Self Weight	160 kN
Slab Weight	85 kN	Slab Weight	100 kN
Roof Weight	125 kN	**Total DL**	260 kN
Total DL	330 kN	SIDL	110 kN
SIDL	155 kN	**Train Load**	190 kN
Crowd Load	80 kN	Braking + Tractive Load	29 kN
LL on Roof	160 kN	Long Welded Rail Forces	58 kN
Total LL	240 kN	Bearing Load	20 kN
Roof Wind Load	85 kN	**Temperature Load**	
Lateral	245 kN	For Track Girder	20 kN
Bearing Load	14 kN	For Platform Girder	14 kN
		Derailment Load	80 kN/m

The force-based design is carried out for Pier as per IS 1893:2002 and IRS CBC 1997 Code and the results are shown in Table 3.3. From the FBD, it is found out that the minimum required cross section of the pier is only 1.5 m x 0.7 m for 2 % reinforcement. The base shear of the pier is 891 kN.

3.2 Design of Pier Using Direct Displacement Based Design

Displacement Ductility	Drift Limit (m)	Cross Section (m)	Base Shear V_b (kN)	Diameter of Bar (mm)	Number of Bars	% of Reinforcement Required
1	0.276	1.5 x 0.7	604	32	#16	1.2 %
2	0.276	1.5 x 0.7	150	32	#12	0.8 %
3	0.276	1.5 x 0.7	86	32	#12	0.8 %
4	0.276	1.5 x 0.7	60	32	#12	0.8 %

The parametric study is carried to know the effect of displacement ductility on base shear for different Performance levels and the results are shown in Figure 3.2. The figure shows that as the displacement ductility level increases the base shear of the pier decreases and also the difference between different performance levels is about 40 %.



3.4 Performance Assessment

The performance assessment is done to study the performance of designed pier by Force Based Design Method and Direct Displacement Based Design Method. For this purpose, Non-linear static analysis is conducted for the designed pier using SeismoStruct Software and the results are shown in Table 3.5. The section considered is 1.5 m x 0.7 m. Performance parameters behaviour factor (R´), structure ductility (μ') and maximum structural drift (Δ'max) are found for both the cases.

The behaviour factor (R´) is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure. The behaviour factor, R´, therefore accounts for the inherent ductility, over the strength of a structure and difference in the level of stresses considered in its design. FEMA 273 (1997), IBC (2003) suggests the R factor in force-based seismic design procedures. It is generally expressed in the following form taking into account the above three components,

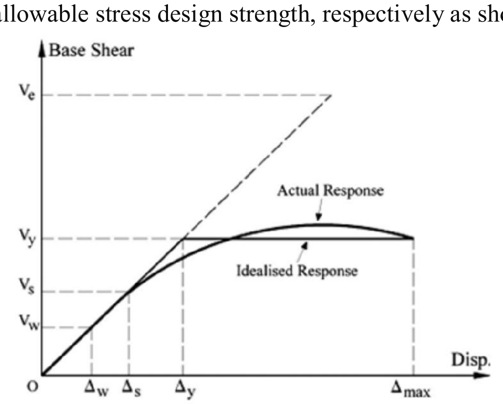
$$R' = R_{\mu} \bullet R_s \bullet Y$$

$$R_{\mu} = \frac{V_e}{V_y}, R_s = \frac{V_y}{V_s}, Y = \frac{V_s}{V_w}$$

where, Rμ is the ductility dependent component also known as the ductility reduction factor, RS is the over-strength factor and Y is termed the allowable stress factor. With reference to Figure 13, in which the actual force–displacement response curve is idealized by a bilinear elastic–perfectly plastic response curve, the behavior factor parameters may be defined as

$$R'(R_w) = \left(\frac{V_e}{V_y}\right)\left(\frac{V_y}{V_s}\right)\left(\frac{V_s}{V_w}\right) = \frac{V_e}{V_w}$$

where, Ve, Vy, Vs and Vw correspond to the structure's elastic response strength, the idealised yield strength, the first significant yield strength and the allowable stress design strength, respectively as shown in the Figure



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The structure ductility, μ', is defined in as maximum structural drift (Δ′max) and the displacement corresponding to the idealised yield strength (Δy) as:

$$\mu' = \frac{\Delta'_{max}}{\Delta_y}$$

In Force Based Design, a force reduction factor (R) of 2.5 is used, and the design base shear is estimated to be 891kN in the FBD. The performance parameters of the section designed using FBD shows that the behavior factor R is found to be about 2.74. The same pier is designed using a DDBD method for target displacement ductility and drift, the performance parameters structural ductility and structural drift are found out for these cases. It shows that the achieved performance parameters are higher than assumed in the design stage in both cases of DDBD. Though the FBD may not always guarantee the performance parameter required, in the present case the pier achieves the target requirement. In the case of DDBD, the design considers the target displacement ductility and drift at the design stage, and the present study shows that in both the examples the DDBD method achieves the behavior factors more than targeted Values. These conclusions can be considered only for the selected pier. For General conclusions large number of case studies is required and it is treated as a scope of future work.

Designed			Type of design	V_b (kN)	% of Steel	Φ (mm)	No. of Bars	Performance Parameters Achieved		
μ	Δ	R						μ	Δ	R
		2.5	FBD	891	2 %	32	#28			2.74
1	0.276		DBD	604	1.2 %	32	#16	3.5	0.35	3.25
2	0.276		DBD	150	0.8 %	32	#12	3.4	0.34	11.63

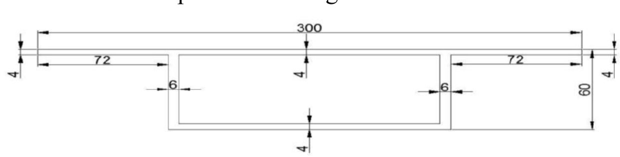
IV. PARAMETRIC STUDY ON BEHAVIOUR OF CURVED BOX GIRDER BRIDGES

Parametric study of box girder bridges using finite element method is described in this chapter. The parameters of box girder bridges considered in this study are radius of curvature, span length, span length to the radius of curvature ratio and number of boxes. The various responses parameters considered are the longitudinal stress at the top and bottom, shear, torsion, moment, deflection and fundamental frequency.

4.1 Validation of the Finite Element Model

To validate the finite element model of box girder bridges in SAP 2000, a numerical example from the literature (Gupta et al., 2010) is considered. Figure shows the cross section of simply supported Box Girder Bridge considered for validation of finite element model. Box girder considered is subjected to two concentrated loads (P = 2 X 800 N) at the two webs of mid span. Span Length assumed in this study is 800 mm and the material property considered are Modulus of elasticity (E) =2. 842GPa and Modulus of rigidity (G) =1. 015GPa.

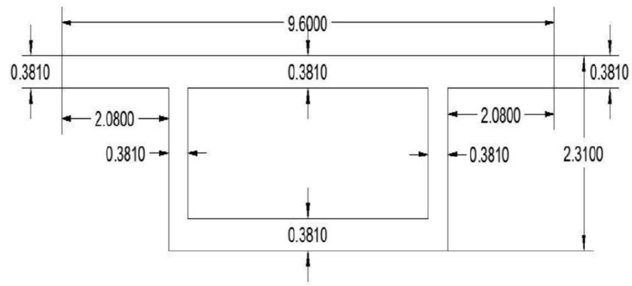
The mid span deflection of the modelled box girder bridge is compared with the literature and it is presented in the Table. From the Table 6, it can be concluded that the present model gives the accurate result.



Parameter	Gupta et al. (2010)	Present Study
Mid Span Deflection (mm)	4.92	4.91

4.2 Case Study of Box Girder Bridges

The geometry of Box Girder Bridge considered in the present study is based on the design basis report of the Bangalore Metro Rail Corporation (BMRC) Limited. In this study, 60 numbers of simply supported box girder bridge model is considered for analysis to study the behaviour of box girder bridges. The details of the cross section considered for this study is given



Span Length (m)	Radius of Curvature (m)	Theta (radian)	Number of Boxes
Radius of Curvature			
31	∞	0.0000	1,2,3
31	100	0.3100	
31	150	0.2067	
31	200	0.1550	
31	250	0.1240	
31	300	0.1033	
31	350	0.0886	
31	400	0.0775	
Span Length			
16	120	0.1333	1,2,3
19	120	0.1583	
22	120	0.1833	
25	120	0.2083	
28	120	0.2333	
31	120	0.2583	
Span Length to Radius of Curvature Ratio			
12	120	0.1000	1,2,3
24	120	0.2000	
36	120	0.3000	
48	120	0.4000	
60	120	0.5000	
72	120	0.6000	

Geometries of Bridges used in Parametric Study

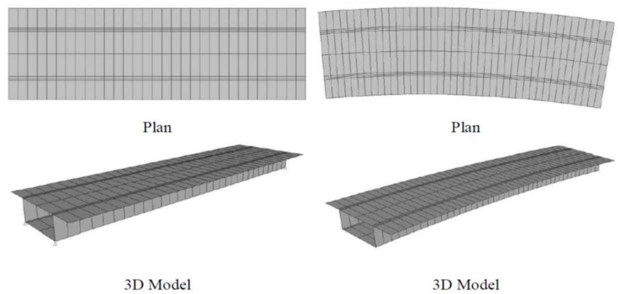
Properties of Material	Value
Weight per unit volume	235400 N/m^3
Mass per unit volume	24000 N/m^3
Modulus of Elasticity (E)	32500 x 10^6 N/m^2
Poisson's Ratio (υ)	0.15
Coefficient of thermal expansion (A)	1.170 x 10^{-5} / °C
Shear Modulus (G)	1.413 x 10^{10} N/m^2
Specific Concrete Compressive Strength (f_c')	45 x $10^6 N/m^2$

Material Properties

The moving load analysis is performed for live load of two-lane IRC 6 Class A (Tracked Vehicle) loading for all the cases considered by using SAP 2000. The longitudinal stress at the top and bottom, shear, torsion, moment, deflection and fundamental frequency is calculated and compared with Single Cell Box Girder (SCBG), Double Cell Box Girder (DCBG) and Triple Cell Box Girder (TCBG) bridge cases for various parameters viz., radius of curvature, span length, and span length to the radius of curvature ratio.

4.3 Finite Element Modelling

The finite element modelling methodology adopted for validation study is used for the present study. The modelling of Box Girder Bridge is carried out using Bridge Module in SAP 2000. The Shell element is used in this finite element model to discretize the bridge cross section. At each node it has six degrees of freedom: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes. The typical finite element discretized model of straight and curved simply supported box Girder Bridge in SAP 2000 is shown in figure



4.4 Parametric Study

The parametric study is carried out to investigate the behaviour (i.e., the longitudinal stress at the top and bottom, shear, torsion, moment, deflection and fundamental frequency) of box girder bridges for different parameters viz. radius of curvature, span length, span length to radius of curvature ratio and number of boxes.

4.5 Radius of Curvature

Two lane 31 m Single Cell Box Girder (SCBG), Double Cell Box Girder (DCBG) and Triple Cell Box Girder (TCBG) Bridge are analyzed for different radius of curvatures to illustrate the variation of longitudinal stresses at the top and bottom, shear, torsion, moment, deflection and fundamental frequency with radius of curvature of box girder bridges.

To express the behavior of box girder bridges curved in plan with reference to straight one, a parameter α is introduced. α is defined as the ratio of response of the curved box girder to the straight box girder.

The variation of longitudinal stress at top with radius of curvature of box girder bridges is shown in Figure17. As the radius of curvature increases, the longitudinal stress at the top side of the cross section decreases for each type of Box Girder Bridge. Variation of Stress between radius of curvature 100 m and 400 m is only about 2 % and it is same for all the three cases.

4.6 Span Length

Two lanes with 120 m radius of curvature Single Cell Box Girder Bridge (SCBG), Double Cell Box Girder Bridge (DCBG) and Triple Cell Box Girder Bridge (TCBG) are analyzed for different span length to illustrate the variation of longitudinal stresses at the top and bottom, shear, torsion, moment, deflection and fundamental frequency with a span length of box girder bridges.

4.7 Span Length to Radius of Curvature Ratio

Two lanes with 120 m radius of curvature Single Cell Box Girder Bridge (SCBG), Double Cell Box Girder Bridge (DCBG) and Triple Cell Box Girder Bridge (TCBG) are analysed for different span length to the radius of curvature of ratio to illustrate the variation of longitudinal stresses at top and bottom, shear, torsion, moment, deflection and fundamental frequency with a span length of box girder bridges.

The variation of Longitudinal Stress at the top with span length to the radius of curvature of ratio of box girder bridges is shown in Figure 38 As the span length to the radius of curvature of ratio increases, longitudinal stress at the top of box girder increases for each type of Box Girder Bridge. Variation of longitudinal stress at the top of box girder between span length to the radius of curvature of ratio 0.1 – 0.6 is about 92 % for all the three cases and it shows that effect of span length to the radius of curvature of the ratio on longitudinal stress at the top is significant. Variation of longitudinal stress at top between three types of box girder is only about 1 %.

V. INTRODUCTION TO PILE FOUNDATIONS

Pile foundations are the part of a structure used to carry and transfer the load of the structure to the bearing ground located at some depth below ground surface. The main components of the foundation are the pile cap and the piles. Piles are long and slender members which transfer the load to deeper soil or rock of high bearing capacity avoiding shallow soil of low bearing capacity the main types of materials used for piles are Wood, steel and concrete. Piles made from these materials are driven, drilled or jacked into the ground and connected to pile caps. Depending upon type of soil, pile material and load transmitting characteristic piles are classified accordingly. In the following chapter we learn about, classifications, functions and pros and cons of piles.

5.1 Function of piles

As with other types of foundations, the purpose of a pile foundations is: to transmit a foundation load to a solid ground to resist vertical, lateral and uplift load A structure can be founded on piles if the soil immediately beneath its base does not have adequate bearing capacity. If the results of site investigation show that the shallow soil is unstable and weak or if the magnitude of the estimated settlement is not acceptable a pile foundation may become considered. Further, a cost estimate may indicate that a pile foundation may be cheaper than any other compared ground improvement costs. In the cases of heavy constructions, it is likely that the bearing capacity of the shallow soil will not be satisfactory, and the construction should be built on pile foundations. Piles can also be used in normal ground conditions to resist horizontal loads. Piles are a convenient method of foundation for works over water, such as jetties or bridge piers.

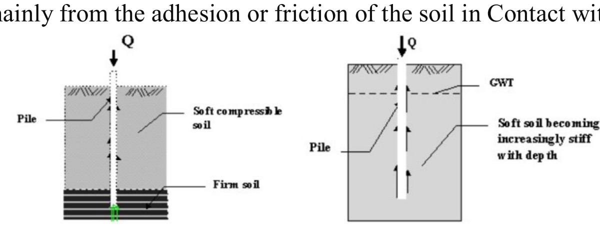
5.2 Classification of Piles

A. End Bearing Piles

These piles transfer their load on to a firm stratum located at a considerable depth below the base of the structure and they derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile (see figure 45). The pile behaves as an ordinary column and should be designed as such. Even in weak soil a pile will not fail by buckling and this effect need only be considered if part of the pile is unsupported, i.e. if it is in either air or water. Load is transmitted to the soil through friction or cohesion. But sometimes, the soil surrounding the pile may adhere to the surface of the pile and causes "Negative Skin Friction" on the pile. This, sometimes have considerable effect on the capacity of the pile. Negative skin friction is caused by the drainage of the ground water and consolidation of the soil. The founding depth of the pile is influenced by the results of the site investigate on and soil test.

B. Friction or Cohesion Piles

Carrying capacity is derived mainly from the adhesion or friction of the soil in Contact with the shaft of the pile



C. Cohesion Piles

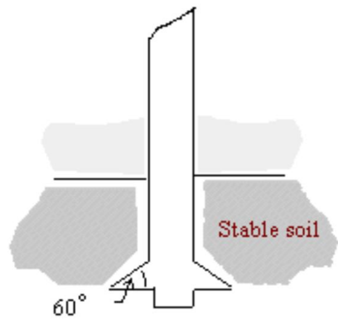
These piles transmit most of their load to the soil through skin friction. This process of driving such piles close to each other in groups greatly reduces the porosity and compressibility of the soil within and around the groups. Therefore, piles of this category are sometimes called compaction piles. During the process of driving the pile into the ground, the soil becomes molded and, as a result loses some of its strength. Therefore, the pile is not able to transfer the exact amount of load which it is intended to immediately after it has been driven. Usually, the soil regains some of its strength three to five months after it has been driven.

D. Friction piles

These piles also transfer their load to the ground through skin friction. The process of driving such piles does not compact the soil appreciably. These types of pile foundations are commonly known as floating pile foundations

E. Combination of Friction Piles and Cohesion Piles

An extension of the end bearing pile when the bearing stratum is not hard, such as a firm clay. The pile is driven far enough into the lower material to develop adequate frictional resistance. A farther variation of the end bearing pile is piles with enlarged bearing areas. This is achieved by forcing a bulb of concrete into the soft stratum immediately above the firm layer to give an enlarged base. A similar effect is produced with bored piles by forming a large cone or bell at the bottom with a special reaming tool. Bored piles which are provided with a bell have a high tensile strength and can be used as tension piles



VI. ULTRA-HIGH-PERFORMANCE CONCRETE

Ultra-High-Performance Concrete is a result of the search for a concrete with a higher strength. Conventional concrete is currently still the most used concrete, but High-Performance Concrete as well as Ultra High-Performance Concrete have a high potential of application and are utilized more and more. Considering the compressive strength, the following division can be made between the three types of concrete:

Conventional concrete up to C53/65
High Performance Concrete C53/65 to C90/105
Ultra-High-Performance Concrete C90/105 to C200/230

The creation of Ultra High-Performance Concrete is made possible by changing the design of the concrete mix by:

- Improving the homogeneity, by using small sized particles the stress variation decreases. This also reduces the transverse tensile stresses. A more homogeneous material result in a more homogeneous stress distribution and thus in a generally stronger material.
- Increasing the package density, by filling the voids with fine particles which can contribute to the strength.
- Improving the microstructure, by hardening the concrete at higher temperatures and/or by hardening at higher atmospheric pressures.
- Adding steel fibres, this leads to small crack distances and give the material large ductility.
- Reducing the water-cement ratio, this way there are less voids created and filled with water, which increases the material strength.

6.1 Reasons for using UHPC

A. Advantages

- More slender structures are possible which reduces the weight.
- The very dense material structure results in a high durability and a smaller concrete cover.
- A higher prestressing is possible.
- The stress loss in prestressing steel is less as there is less shrinkage.
- It is possible to construct without steel reinforcement.

B. Disadvantages

- UHPC with steel fibres cannot be recycled as the steel fibres can hardly be taken out.
- UHPC is more expensive than conventional concrete
- The production capacity of a concrete mixing plant decreases for the production of UHPC as the mixing takes

longer and is more complicated.

- The hydration process within UHPC is fast which results in a large heat production. This results in a fast hardening shrinkage during the first days. Variable temperatures during hardening within the concrete result in internal stresses causing cracks. It is therefore important to control the hardening process and protect it from the changing weather conditions. This is particularly important for thick construction elements.
- There is little known about fatigue of the material.

6.2 Mix Procedure

The mix design of UHPC is complex. With a low water-cement ratio and many additives the mix is relatively dry. This makes it hard to mix all the materials together. By adding superplasticizers, the workability of the mix can be ensured. It is important to divide the additives and fillers into doses to the mix and at the right time. This however results in a longer mixing time. The UHPC composition is very sensitive to little variations in the amount of materials and is also influenced by the weather conditions.

The latter has less influence when the mixing takes place in factories. Considering all this it is assumed best to utilize precast UHPC elements instead of in-situ UHPC as this ensures a better quality. However, the utilization of in-situ UHPC is not impossible. Special attention should be paid to the curing of the UHPC because of its very low or even total absence of bleeding. The outer skin and construction joints should be checked and cured to prevent drying out of the concrete causing microcracks

VII. FIBER REINFORCED POLYMERS

Fibre Reinforced Polymer (FRP) is a composite material. This means the combination of two or more materials on a macroscopic scale to form a useful material. By combining fibres with polymer resins a material is created with characteristics that none of the components exhibit independently.

7.1 Reasons for using FRP

Standard properties of FRP composites:

- High strength at low weight
- Good impact, compression, fatigue and electrical properties
- Ability to fabricate massive one-piece mouldings
- Moulding to close dimensional tolerances
- Short installation times

The following additional properties can readily be provided by reinforcement and/or matrix alteration, chemical addition or other formulation, material or fabrication alteration:

- Excellent chemical and corrosion resistance
- High ultraviolet radiation stability
- Good-to-excellent fire hardness
- Good structural integrity
- Good thermal insulation
- Ability to attenuate sound
- Respectable abrasion resistance
- Ready bonding to dissimilar materials
- Medium-to-high productivity rates

The physical, mechanical and cost-effective properties of any reinforced plastic composite can be ‘tailored’ over a wide range to suffice with the performance specification demanded. This however has the disadvantage that there are few ‘standard’ composites. Besides, companies which fabricate FRP often keep their product information secret. This has led to few codes for specific applications. The key to a more widespread use of FRP materials is to have manufacturer-independent application codes for civil engineering practice. The problem of the wide variety of materials and possibilities of application could be overcome by their classification in so-called Application Categories, for which in a first step application recommendations could be worked out.

Several other disadvantages of FRP are:

- Poor ductility, particularly when compared with metals and considering those composites that are thermoset-based
- Low stiffness in comparison to many traditional and/or competitive materials
- Temperature is limited, with few exceptions, not above150 °C
- Limited recycling ability even when thermoplastic-based
- Material costs still high

7.2 Materials

FRP consist of load-bearing fibres and a resin matrix in which they are embedded. Whereas the fibers exercise the actual load-bearing function, the polymer matrix essentially has four functions:

- Fixing the fibres in the desired geometrical arrangement
- Transferring the forces to the fibres
- Preventing buckling of the fibres under compression actions
- Protecting the fibers from humidity etc.

The mechanical properties of fibre-polymer bonds are mainly determined by the adhesion and the mechanical compatibility between the fibres and the matrix as well as the angle between the fibers and the direction of loading. In order to obtain a good mechanical interaction between the fibers and the matrix, their parameters must be adjusted to each other.

7.3 Reinforcements

Reinforcements are used with resin systems to improve the mechanical properties of cured2 resin and provide usable components. By far the most important fibre used with epoxy resins is glass fiber, which is supplied in a variety of forms. In recent years high strength carbon fibers and polyaramid fibers are used increasingly in the manufacturing of composite materials for all applications, often in the form of hybrid products where the best features of each constituent are utilized to the full. The high strength and stiffness-to-weight ratios of carbon and polyaramid fibers make them particularly attractive for the manufacture of lightweight structural components. The costs are however far more expensive compared with glass fibres, with polyaramid fibres about 15 and carbon fibers about 40 time the cost of glass fibres. The proportion of reinforcement present in a composite has a major effect on its properties, so does fibre type and fibre orientation. Fiber content is usually expressed in terms of a weight fraction, a volume fraction or a resin/fiber ratio

7.4 Resins

On their own, bundles of parallel fibres are of little use in a load-bearing structure. They may have structural integrity in tension but unless they can be joined, their structural potential cannot be harnessed. Similarly, bundles of fibres are almost useless in shear and compression. Without a means of distributing load across a series of fibre bundles, the material is of no use for structural applications other than rope, which works in cable tension. By 'gluing' bundles of fibers together with resin matrices, materials are made where the strong, stiff fibres are able to carry most of the stress whilst the matrix distributes the external load to all the fibres as well as providing protection and preventing fiber buckling under compressive loads. The most critical region for the load transfer process is the fiber-resin interface. Common applied resins in FRP are:

- Polyester Resins
- Vinyl Ester Resins
- Epoxy Resins

Epoxy resins offer the best mechanical properties for FRP. The costs of epoxies are higher than polyesters but they are very versatile and widely used.

7.5 Cores

Sandwich construction is commonly used with composites to increase structural efficiency, with the FRP forming the outer skins and bonded to a variety of core materials.

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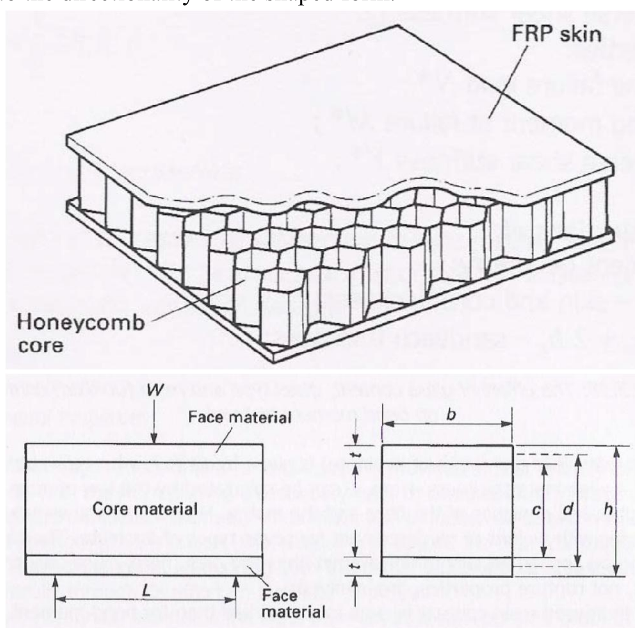
Materials available as cores are in three forms:

1. Basically lightweight (e.g. balsa wood)
2. Lightweight because foamed
3. Lightweight because honeycombed.

Bridge deck systems and bridge structures made of FRP are often sandwich elements with a core of honeycombs or something alike Figure. Honeycombs are formed from in fact any thin-sheet material connected together in a manner that resembles the honeycomb made by bees hence the name. The structural performance is particularly high in direct compression and shear due to the directionality of the shaped form.



7.6 Bridges

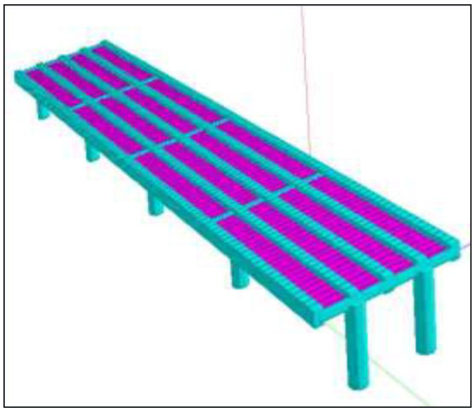
Bridges made of FRP are applied increasingly the last years. Despite the high material costs, the lack of experience with respect to durability and long-term behaviour and the lack of standards, application guidelines and design codes, it becomes gradually more attractive to construct with this material. The bridges made of FRP are often sandwich superstructures and serve as footbridges and smaller highway bridges. Cable stayed and truss bridges made of FRP can also be found but are rarer. In the future it is expected that FRP will become more economical which means a broader application range. It is expected that FRP bridges will then be constructed for more applications.

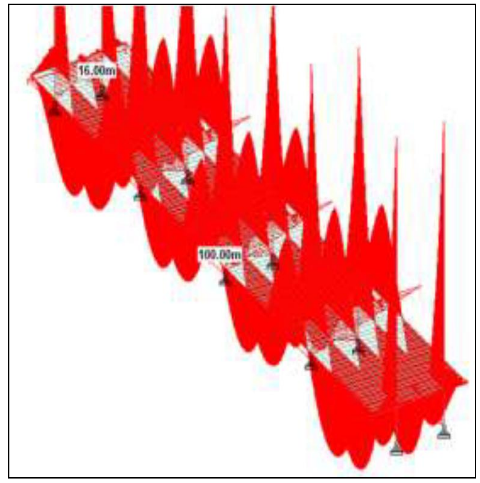
VIII. EXPERIMENT: DESIGN OF METRO RAIL BRIDGE BY PIER USING FORCE BASED DESIGN THROUGH STAAD PRO

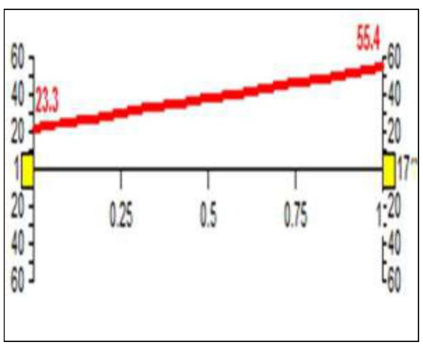
This chapter presents case studies on design of metro rail bridge by pier using force-based design through STAAD pro in Techture Structures Private Limited Indore, which are used to illustrate the developed methodology for design of metro rail bridge by pier using force-based design through STAAD pro. The case study materials were collected from the particular construction projects in Techture Structures Private Limited.

8.1 Methodology Design of Pier Using Force Based Design

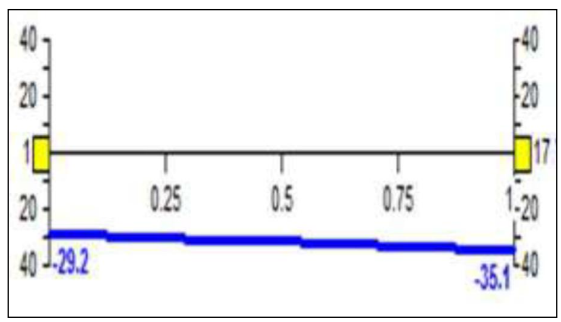
STAAD. Pro. in space is Operated with units Meter and Kilo Newton. The geometry is drawn and the section properties are assigned. Fixed Supports are taken. Quadrilateral meshing is done followed by assigning of plate thickness.3D rendering can be viewed for the geometry. Loads are defined by the loads and definitions. By Post Processing mode, Nodal displacement, Max. Absolute Stress distribution for the bridge can be viewed. Run analysis is operated. Max. Response by the IRC Class 70R loading is done by STAAD.beava. The deck is created in bridge deck processor, this being the first step of STAAD.beava. In STAAD.beava, roadways, curbs, vehicular parameters are provided. Lastly transfer of load is done into STAAD Pro. For further analysis and design. All the Max response criteria are checked Mx,My,Mz stresses etc for different members elements. The load positions and reactions, beam forces and moments. are determined. The concrete is designed as per IS Code.



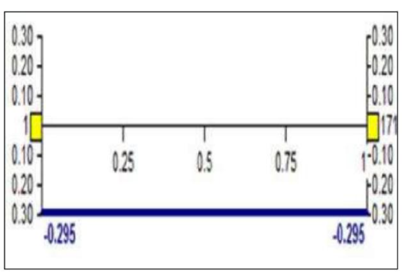




Mz(kNm) Beam Graph



Fy(kN) Beam Graph



Fx(kN) Beam Graph

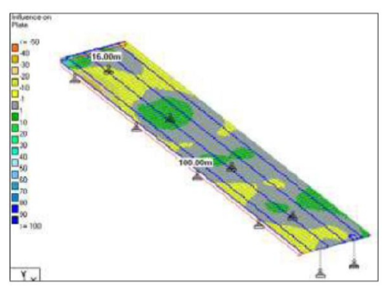


Plate Stresses

IX. RESULTS AND DISCUSSIONS

The output data for the IRC Class 70R bogie loadings are considered which include nodal displacement, nodal displacement summary, beam forces, beam end displacements, beam end displacement summary, reactions, reaction summary, axial forces, beam moments, live load effect and many more by STAAD. Pro V8i. As all of them cannot be described in this project, the data result tables being very large, some of the glimpse of the output results in the tabular forms is provided in this below.

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9.1 Vehicle Loading

The loading vehicle details are given:
Design Code = IRC
Loading Class = Class 70R Loading
Max. Effect = 9.39626m
Unit of Length = m
Unit of Force = kN
Combination Factor =1
No. of Traffic Lanes = 6
Traffic Lane number 1
Lane Factor = 1
The loading vehicle details are
Width = 2900
Front Clearance = 31675
Rear Clearance = 31675
No. of Axles = 3
Vehicles travel in the roadway direction

Vehicle No.	Position x	Position y	Orientation
1	17.171	0.05	0

End Lane
Traffic Lane No. 2
End Lane
Traffic Lane No. 3
Lane Factor 1

The loading vehicle details are
Width = 2900
Front Clearance = 31675
Rear Clearance = 31675
No. of Axles = 3
Vehicles travel in the roadway direction

Vehicle No.	Position x	Position y	Orientation
1	11.9501	88.219	1.5708
2	11.9501	49.689	1.5708
1	12.05	-4.35305	1.5708

End Lane
Traffic Lane No. 4
Lane Factor 1

The loading vehicle details are
Width = 2900
Front Clearance = 31675
Rear Clearance = 31675

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No. of Axles = 3
Vehicles travel in the roadway direction

Vehicle No.	Position x	Position y	Orientation
1	8.0501	97.7264	1.5708
2	8.05005	50.1894	1.5708
1	7.95	-2.85188	1.5708

End Lane
Traffic Lane No. 5
Lane Factor 1
The loading vehicle details are
Width = 2900
Front Clearance = 31675
Rear Clearance = 31675
No. of Axles = 3
Vehicles travel in the roadway direction
End Lane

Traffic Lane No. 6
Lane Factor 1

Vehicle No.	Position x	Position y	Orientation
1	3.95	99.72	1.5708
2	3.95	49.689	1.5708
1	4.05	0.65	1.5708

It cuts time and gives safe values required for its design. By this approach of design, maximum loads created by STAAD. behava are transferred into STAAD Pro. and the analysis and design is then carried out.
The loading vehicle details are
Width = 2900
Front Clearance = 31675
Rear Clearance = 31675
No. of Axles = 3
Vehicles travel in the roadway direction

Vehicle No.	Position x	Position y	Orientation
1	-1.74	88.71	1.5708
2	-1.74	50.18	1.5708
1	-4.43	-4.35305	1.5708

9.2 Concrete Design Details

The concrete is designed for element no.
61 which gives the following result:
For
FY: 413.682MPA;
FC: 27.579MPA;
Cover (top):19.05mm;
Cover (bottom): 19.05mm
Longitudinal Direction-only minimum steel required;
Transverse Direction – only minimum steel required;

LONG.REINF Sq.mm/mm	MOM-X/LOAD Kn-mm/mm	Trans.reinf Sq.mm/mm	OM-Y/LOAD Kn-mm/mm
TOP-0.540	24.16/2	0.540	0
BOTTOM-0.545	54.76/1	0.782	1

X. SUMMARY AND CONCLUSIONS

10. Summary

A metro system is an electric passenger railway transport system in an urban area with a high capacity, frequency and the grade separation from other traffic. An elevated metro system is the most preferred form of metro structure due to ease of construction and less cost compared to other types of metro structures. An elevated metro system has two major components pier and box girder. In this project, study has been carried out on these two major elements.

In the first part of this study, the performance assessment on designed pier by Force Based Design and Direct Displacement Based Design is carried out. The design of the pier is done by both force-based design method and direct displacement-based design method.

In the second part, parametric study on behaviour of box girder bridges is carried out by using finite element method. The numerical analysis of finite element model is validated with model of Gupta et al. (2010). The parameter 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 response parameter 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.

XI. CONCLUSION

The performance assessment of selected designed pier showed that,

- Force Based Design Method may not always guarantee the performance parameter required and in the present case the pier just achieved the target required.
- In case of Direct Displacement Based Design Method, selected pier achieved the behavior factors more than targeted Values.

These conclusions can be considered only for the selected pier. For General conclusions large numbers of case studies are required and it is treated as a scope of future work.

The parametric study on behavior of box girder bridges showed that,

- As the radius of curvature increases, responses parameter longitudinal stresses at the top and bottom, shear, torsion, moment and deflection are 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.
- As the span length increases, responses parameter longitudinal stresses at the top and bottom, shear, torsion, moment and deflection are increases for three types of box girder bridges and fundamental frequency decreases for three types of box girder bridges.
- 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 and as span length to the radius of curvature ratio increases fundamental frequency decreases for three types of box girder bridges.

The conclusions considering the dimensions of the elevated metro structure together with useful information from chapter 6 and 7. The design study concerns the structural design of the elevated metro system. Different concepts will be analyzed for the elevated railway structure made of conventional concrete and Ultra High-Performance Concrete (UHPC) and the best concept will be further elaborated. Besides a global design for the elevated railway structure made of Fiber Reinforced Polymers (FRP) will be made.

- Analysis and design of the elevated Metro Bridge as per IRC codes (here IRC 70R loading) can be easily done by STAAD.Pro. in connection with STAAD.beava. mechanism is well understood.
- The maximum resultant nodal displacement is for node 1529; 0..015mm in x, -51.203mm in y and -.287mm in x.
- The maximum resultant beam end displacement is for beam 1930 and node 1529 equivalent to 51.204.
- The maximum and minimum values for beam maximum forces by section property are computed for axial, shear and bending.
- The effect of vertical loading for 6 traffic lanes showing width, front clearance, rear clearance, no. of axles, positon in x, position in y with orientation can be determined. The orientation varies from 0 to 1.5708.
- The concrete design for element 61 gives the top and bottom longitudinal reinforcement is 0.540 and 0.545. The top and bottom transverse reinforcement are 0.540 and 0.780 for element 61. Similarly, for another element, it can be found out.
- It is must for today's engineers, designers, research scholars to make an effective contribution to what is the purpose of each high-quality design and for the improvement of quality of environment in which we all are residing. Thus, evolution of software must be properly used so that it meets the beneficiary needs.

Predominant initiators of industrial solid wastes are the thermal power plants producing coal ash, the integrated Iron and Steel mills producing blast furnace slag and steel melting slag, non-ferrous industries like aluminum, zinc and copper producing red mud and tailings, sugar industries generating press mud, pulp and paper industries producing lime and fertilizer and allied industries producing gypsum.

REFERENCES

[1]. Abdelfattah, F. A. (1997). Shear lag in steel box girders. Alexandria Eng. J., Alexandria Univ., Egypt, 36 (1), 1110–1118.
[2]. Armstrong, W. L. and Landon, J. A. (1973). Dynamic testing of curved box beam bridge. Fed. Hwy. Res. and Devel. Rep. No. 73-1, Federal Highway Administration, Washington, D.C.
[3]. Balendra, T. and Shanmugam, N. E. (1985). Vibrational characteristics of multicellular structures. J. Struct. Engrg. , ASCE, 111 (7), 1449-1459.
[4]. Bazant, Z. P. , and El Nimeiri, M. (1974). Stiffness method for curved box girders at initial stress. J. Struct. Div., 100 (10), 2071–2090.
[5]. Buchanan, J. D., Yoo, C. H., and Heins, C. P. (1974). Field study of a curved box-girder bridge. Civ. Engrg. Rep. No. 59, University of Maryland, College Park, Md.
[6]. Chang, S. T., and Zheng, F. Z. (1987). Negative shear lag in cantilever box girder with constant depth. J. Struct. Eng., 113 (1), 20–35.
[7]. Chapman, J. C. , Dowling, P. J. , Lim, P. T. K. , and Billington, C. J. (1971). The structural behavior of steel

and concrete box girder bridges. Struct. Eng., 49 (3), 111–120.

[8]. Cheung, M. S., and Megnounif, A. (1991). Parametric study of design variations on the vibration modes of box-girder bridges. Can. J. Civ. Engrg., Ottawa, 18(5), 789-798.

[9]. Cheung, M. S., and Mirza, M. S. (1986). A study of the response of composite concrete deck-steel box-girder bridges. Proc., 3rd Int. Conf. on Computational and Experimental Measurements, Pergamon, Oxford, 549-565.

[10]. Cheung, M. S., Chan, M. Y. T., and Beauchamp, T. C. (1982). Impact factors for composite steel box-girder bridges. Proc., Int. Assn. for Bridges and Struct. Engrg. IABSE Colloquium, Zurich, 841-848.

[11]. Abdelfattah, F. A. (1997). Shear lag in steel box girders. Alexandria Eng. J., Alexandria Univ., Egypt, 36 (1), 1110– 1118.

[12]. Armstrong, W. L. and Landon, J. A. (1973). Dynamic testing of curved box beam bridge. Fed. Hwy. Res. and Devel. Rep. No. 73-1, Federal Highway Administration, Washington, D.C.

[13]. Balendra, T. and Shanmugam, N. E. (1985). Vibrational characteristics of multicellular structures. J. Struct. Engrg. , ASCE, 111 (7), 1449-1459.

[14]. Bazant, Z. P. , and El Nimeiri, M. (1974). Stiffness method for curved box girders at initial stress. J. Struct. Div., 100 (10), 2071–2090.

[15]. Buchanan, J. D., Yoo, C. H., and Heins, C. P. (1974). Field study of a curved box-girder bridge. Civ. Engrg. Rep. No. 59, University of Maryland, College Park, Md.

[16]. Chang, S. T., and Zheng, F. Z. (1987). Negative shear lag in cantilever box girder with constant depth. J. Struct. Eng., 113 (1), 20–35.

[17]. Chapman, J. C. , Dowling, P. J. , Lim,

[18]. P. T. K. , and Billington, C. J. (1971). The structural behavior of steel and concrete box girder bridges. Struct. Eng., 49 (3), 111–120.

[19]. Cheung, M. S., and Megnounif, A. (1991). Parametric study of design variations on the vibration modes of box-girder bridges. Can. J. Civ. Engrg., Ottawa, 18(5), 789-798.

[20]. Cheung, M. S., and Mirza, M. S. (1986). A study of the response of composite concrete deck-steel box- girder bridges. Proc., 3rd Int. Conf. on Computational and Experimental Measurements, Pergamon, Oxford, 549-565.

[21]. Cheung, M. S., Chan, M. Y. T., and Beauchamp, T. C. (1982). Impact factors for composite steel box-girder bridges. Proc., Int. Assn. for Bridges and Struct. Engrg. IABSE Colloquium, Zurich, 841-848.

[22]. Cheung, Y. K. , and Cheung, M. S. (1972). Free vibration of curved and straight beam-slab or box-girder bridges. IABSE Periodica, Zurich, 32(2), 41-52