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COMPARATIVE STUDY OF RAILWAY BOX BRIDGE USING STAAD PRO & MDM

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ABSTRACT: This study demonstrates the structural analysis and design of RCC box type bridge using manual approach (i.e. MDM method) and by computational approach (Staad-pro) using IRS - CBC codes. The structural elements (top slab, bottom slab, side wall) were designed to withstand Ultimate Load criteria (maximum bending moment and shear force) due to various loads (Dead Load, Live Load, LL surcharge, DL surcharge) and serviceability criteria (Crack width) and a comparative study of the results obtained from the above two approach has been carried out to validate the correctness of the results. Further, it was also observed that the analysis using manual calculation becomes very tedious and cumbersome and for a complex type of structure, thus it is quite a complex task to perform the analysis manually, so the use of computational method (Staad Pro and excel sheet) becomes the obvious choice for design. The results obtained using MDM method shows a good agreement with the results obtained from computational methods. Bridges are the structural components that are required for the efficient movement of Trains and locomotives and under earth embankment for crossing of water course like streams across the embankment as road embankment cannot be allowed to obstruct the natural water way. Bridges can be of different shapes such as arch, slab and box. These can be constructed with different material such as masonry (brick, stone etc.) or reinforced cement concrete. Since bridge pass through the earthen embankment, these are subjected to same traffic loads as the road carries and therefore, required to be designed for such loads. The cushion depends on rail profile at the bridge location.

KEYWORDS: Railway, Bridge, Locomotive, Structural, Design, Analysis.

1 INTRODUCTION

Bridge construction nowadays has achieved a worldwide level of importance. With rapid technology growth the conventional bridge has been replaced by innovative cost-effective structural system. The efficient dispersal of congested traffic, economic considerations, and aesthetic desirability has increased the popularity of box type bridges these days in modern highway systems, including urban interchanges. They are prominently used in freeway and bridge systems due to its structural efficiency, serviceability, better stability, pleasing aesthetics and economy of construction. They are efficient form of construction for bridges because it minimizes weight, while maximizing flexural stiffness and capacity. It has high torsional stiffness and strength, compared with an equivalent member of open cross section. Although significant research has been underway on advanced analysis for many years to better understand the behavior of all types of box bridges, the results of these various research works are scattered and unevaluated. Hence, a transparent understanding of more recent work on straight and curved box bridges is highly desired which divulged the attention towards aiming a present study. The main objective is to provide a clear vision about the analysis and design of box type minor railway bridges. This study would enable bridge engineers to better understand the behavior of Box Bridge outlining a different approach towards analysis and design.

This study was a part of contract package of Eastern Dedicated Freight Corridor Design and Construction of Civil, Structures and Track works for double line Railway under which a box type bridge of 4.5 m span. Specific details for the design are discussed below:

- The box is having the clear height is 2.25m and top and bottom slab thickness of 0.25m.
- The carriage way of box bridge is of 2 lanes.
- The thickness of wearing coat is 0.065m.
- The minimum soil bearing capacity for RCC box Structures is assumed to be 20kN/m2.
- The design life of a structure is that period for which it shall be designed to fulfil its intended function. The design life of all bridge structures is considered as 100 years.

A box structure with top slab, side wall and bottom slab is shown in Fig. 1 along with the loads and reactions. The top slab is subjected to uniformly distributed loads while the sidewalls are subjected to trapezoidal load varying along the height of the structure. The bottom slab is directly resting on soil and is taken as a spring support.

The structural design involves consideration of load cases (box empty, full, surcharge loads etc.) and factors like live load, effective width, braking force, dispersal of load through fill, impact factor, co-efficient of earth pressure etc. Relevant IRCs are required to be referred.

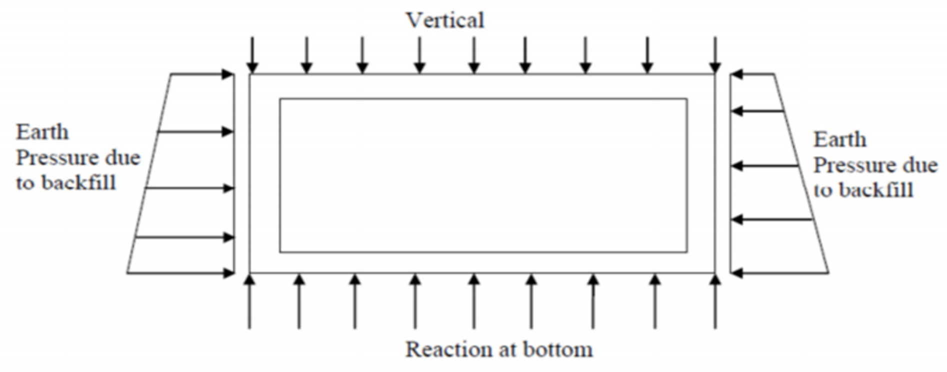


Figure 1. 2-Dimensional model showing width loads and reactions

The structural elements are required to be designed to withstand maximum bending moment and shear force. This paper provides discussions on the provisions in the Codes, considerations and justification of all the above aspects on design. The box bridge can be analyzed either by Software or Computational methods.so it is necessary to study the effectiveness of results obtained from both the methods.

2 METHODOLOGY

Structural analysis incorporates the fields of mechanics and dynamics as well as the many failure theories. From a theoretical perspective, the primary goal of structural analysis is the computation of deformations, internal forces, and stresses.

- Kani’s method
- Slope Deflection Method.
- Moment Distribution Method.

2.1 Kani’s method

It involves distributing the unknown fixed end moments of structural members to adjacent joints, in order to satisfy the conditions of continuity of slopes and displacements. The most important feature of Kani’s method is that it is self-corrective. Any error at any stage of iteration is corrected in subsequent steps. Kani's method was introduced by Gasper Kani in 1940's. Kani's method is also known as Rotation contribution method.

2.2 Slope deflection method

It can be used to analyze statically determinate and indeterminate beams and frames. In slope deflection method, it is assumed that all deformations are due to bending only; influences of axial and shear stresses are ignored. Another assumption is that all the joints of the frame are rigid, i.e., the angles between the members at the joints do not change, when the members of the frame are loaded. As introduced earlier, the slope-deflection method can be used to analyze statically determinate and indeterminate beams and frames. In this method it is assumed that all deformations are due to bending only. In other words, deformations due to axial forces are neglected. The slope deflection method is a structural analysis method for beams and frames introduced in 1914 by George A. Maney. The slope deflection method was widely used for more than a decade until the moment distribution method was developed.

2.3 Moment distribution method

It is a method of successive approximation that may be conducted for any desired degree of accuracy. Basically, the method begins by assuming each joint of a structure fixed. After that, unlocking and locking each joint in succession, the internal moments at joints are distributed and balanced until the joints have rotated to their final position. Moment distribution method offers a convenient way to analyse statically indeterminate beams and rigid frames. In the moment distribution method, every joint of the structure to be analyzed is fixed so as to develop the fixed-end moments. Then, each fixed joint is sequentially released and the fixed-end moments (which by the time of release are not in equilibrium) are distributed to adjacent members until equilibrium is achieved. Moreover, the moment distribution method in mathematical terms can be demonstrated as the process of solving a set of simultaneous equations by means of iteration. Both moment-distribution and slope-deflection methods can be applied to similar types of problems, namely continuous beams and rigid jointed frameworks. Moment distribution has an arithmetical solution in which a number of successive corrections are applied to an initial set of assumed moments.

3 GEOMETRY OF BOX BRIDGE

All the dimensions that have been decided for the designing of a bridge are as follows:

- 1. R.C.C. twin box
    A) No. of Boxes= 2
- Span = 4.5m
- Clear Height = 2.25m
- Top Slab Thickness = 0.25m
- Bottom Slab Thickness = 0.25m
- Side Wall Thickness = 0.25m
- Unit Weight of Concrete = 25kN/m^3
- Unit Weight of Soil = 20kN/m^3
- Modulus of Subgrade of Soil = 250000kN/m^2/m

- Co-efficient of Earth Pressure at Rest = 0.5
- Total Cushion on Top = 0.0m
- Thickness of Wearing Coat = 0.065m
- Carriageway = 2 Lane
- Grade of Concrete = M25
- Grade of Steel = Fe415

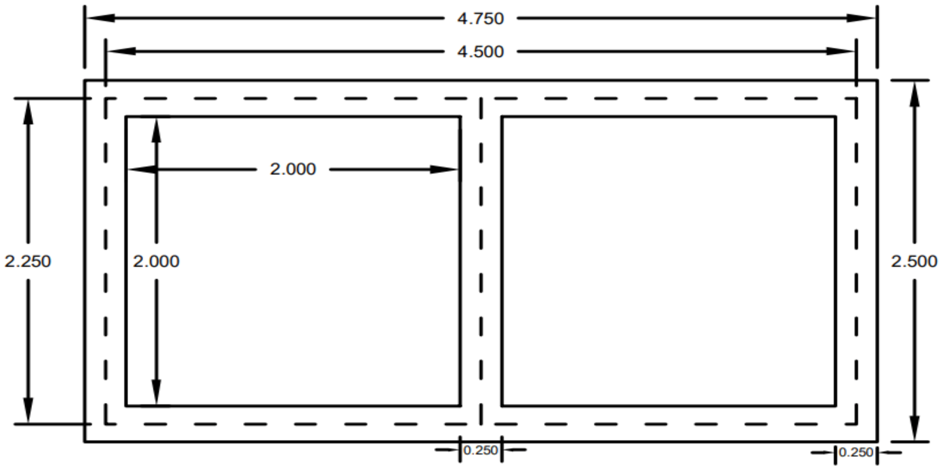


Figure 2. Cross section of bridge

4 CALCULATION BY MDM METHOD

Given data:

1. Span of bridge = 4.5m
2. Clear height = 2.25m
3. Top slab thickness = 0.25 m
4. Bottom slab thickness = 0.25 m
5. Side wall thickness = 0.25 m
6. Unit weight of concrete = 25 KN/m3
7. Unit weight of soil = 20 KN/m3
8. Thickness of wearing coat = 0.065 m

4.1 Load calculations

Total load on top slab= Self wt . Of slab + External dead load + External live load

A) Self wt. of top slab = 0.25 × 25 = 6.25KN/m^2

B) External dead load
   1. Self wt. Of slap = 0.25 ×25 ×1 = 6.25 KN/m^2
   2. Self wt. Of wearing coat = 0.65 × 22 ×1 = 1.43 KN/m^2

3. Dead load = $b \times d \times \gamma = 2.25 \times 0.25 \times 25 = 14.06$ KN/m^2

Total dead load = 14.06 + 6.25 + 1.43

Total dead load = 21.75KN/m^2

C) Calculation of live load
According to IRC, Bridge rule appendix XЦ (a)
Live load = 42.25 KN/m^2

Total load on top slab (w) = 6.25+ 21.75+42.25 = 70.25 kN/m^2

4.2 Coefficient of earth pressure

Ka = $(1 - \sin \phi) / (1 + \sin \phi) = 0.297$

A. Lateral pressure due to (DL + LL)
   Lateral pressure = total vertical load × ka
   = 64 × 0.297
   = 19KN/m2

B. Lateral pressure due to soil = ka × soil × h
   = 0.297 × 20 × 2.25
   = 18.365 KN/m2

Lateral pressure at top = 19 KN/m2
Lateral pressure at bottom = 19+ 13.37 = 32.37 kN/m2

4.3 Distribution factor (df)

Df = K/Ek

Relative Stiffness (k) = I/L

KAD = I/L = 1/2 = 0.5

KAF = KAF = 1/2 × I/L = 1/2 ×1/2 = 0.25

DEAD = KAD / (KAD + KAF) = 0.66

DFAE= KAE / (KAE + KAD) = 0.33

4.4 Fixed end moments

1. MFAB = -29.64 KNm
2. MFDC = 35.96 KNm
3. MFAD = 10.27 KNm
4. MFDA = -11.40 KNm

Table 1. Moment distribution method

Joint	**D**		**A**	
	DC	**DA**	**AD**	**AB**
D.F.	0.33	0.66	0.66	0.33
F.E.M.	35.96	-11.40	10.27	-29.64
Balance	-8.11	-16.21	+12.78	+6.39
Carryover		+6.39	-8.10	
Balance	-2.11	-4.22	5.35	2.67
Carryover		2.68	-2.11	
Balance	-0.89	-1.77	1.39	0.69
Total	74.55	-74.55	58.74	-58.74

4.5 Shear force

A) For horizontal slab AB carrying UDL at 70.25 KN/m2 the reaction at A and B
= 79.03 KN

B) For slab DC, VD=VC
= 95.90 KN

4.6 Net bending moment

1) Free Bending moment at E = (W L^2)/12 = 44.46
Net BM at E = 44.46- 26.56 =28.3KN.m

2) Free Bending moment at F
Net BM at F = 53.98 - 32.37 = 21.61 KN.m

4.7 For vertically member AD

1) Bending moment at midspan = [(W L^2)/8 + (W L^2)/16] = 16.25 KN.m
Net Bending moment at midpoint of AD = 5.98 KN.m

4.8 Design of top slab

1) Bending moment at centre = 28.3KN.m
2) Bending moment at end = 58.74KN.m
3) Direct Force = Ha = 31.24 KN.m
Area = 200 mm2
σst = 200 KN/m2
d = 200 - 50 = 150 mm
Ast = 17.9 x 10 6/ 200 x .9 x 150 = 994.44
Consider 16 mm ϕ
No. of Bars = 994.44/π/4 x 16 x 16 = 5 bars
Spacing = 200 mm^2
Therefore provide 16 mm ¢ bars at 200 mm c/c

Ast provided = 1000 x 3.14 x 16 x 16/200 x 4 = 1005 mm^2
The bars are bent near support at distance = L/5 = .45
Area of distribution steel = .2 % gross section = 0.2 % bD = 400 mm^2
Area of Each faces = 400/2 = 200 mm^2
Consider 8 mm ¢ bars
A = π/4 x 8 x 8 = 50.26 mm^2
No of bars = 200/50.26 = 4 Numbers
Spacing = 1000 x 50.26/200 = 251.3 = 250 mm
Therefore provide 8 mm ϕ bars at 200 mm c/c as reinforcement
BM at support
Ast = m/σst.j.d = 543.89 mm^2
Area available from bent up bars from middle section
= Ast provided/2 = 502.5 mm^2
Provide additional bars of 8 mm at 200 mm c/c

4.9 Design of bottom slab

1) BM at centre = 21.61 KN.m
2) BM at support = 74.55 KN.m.
3) Direct Force = 26.56 KN.m
Dreq = 200 mm
Ast = m/ σst x j x d = 21.61 x 10 6/ 200 x 0.9 x 150 = 1200 mm^2
Consider 18 mm ϕ bars
No of bars = 1200.56 x 4 / π x 18 x 18 = 5 bars
Spacing = 1000 x π x 18 x 18/ 1200.56 x 4 = 200 mm
Therefore provide 5 bars of 18 mm ϕ at 200 mm c/c
Ast provided = 5 x π x 18 x 18/4 = 1272.34 mm^2
Area provided for bar bent towards the centre near support = 1272.34/2 = 636.17 mm^2
Consider 20 mm ¢ bars
Therefore provide 20 mm ¢ bars at 250 mm c/c

4.10 Design of side wall

1) BM at centre = 30.26KN.m
2) BM at End = 10.25KN.m
3) Direct Force = 95.90 KN
Side wall can be designed using sp.16
Mu/fck b d2 = 16.25 x 10 6/ 25 x 1000 x 200 x 200 = 0.029
Fu/fck b D2 = 95.90 x 10 x 10/ 25 x 1000 x 150 = 0.026
Ast = PbD /100 = 0.5 x 1000 x 200 / 100
Ast = 1000 mm^2

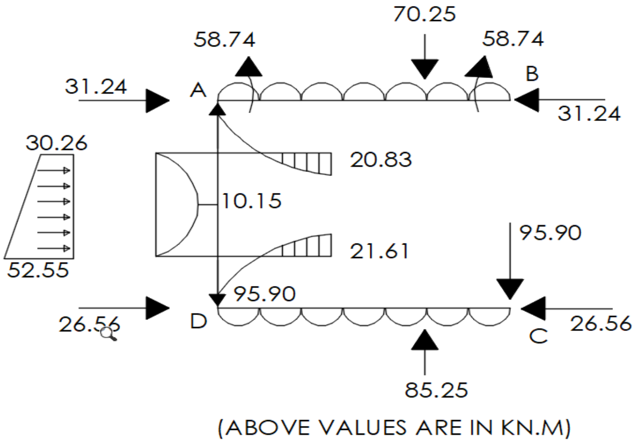


Figure 3. BMD and SFD diagram by MDM method

5 ANALYSIS OF BOX BRIDGE USING STAAD PRO SOFTWARE

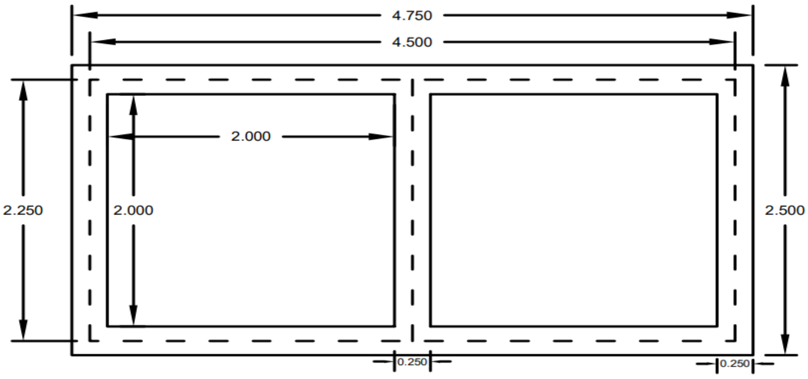


Figure 4. Geometry

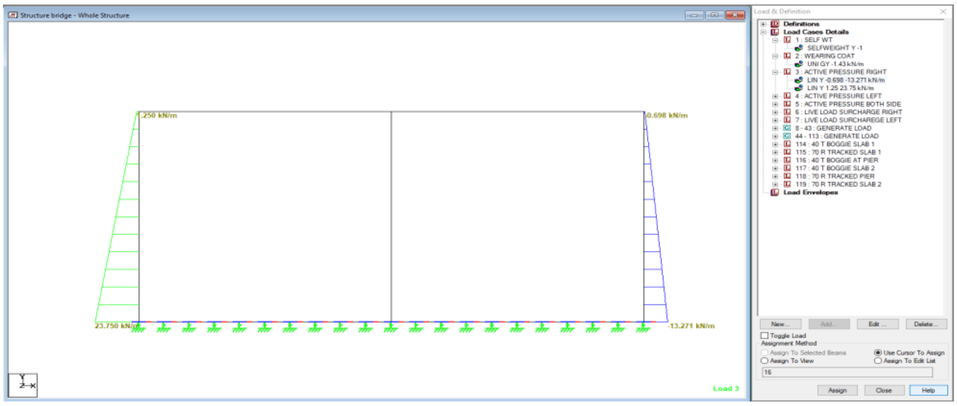
5.1 Apply dead load on Staadpro

Load case detail

1. Self - wt. = -1 (Direction- G(y)).
   Assign to view
2. Wearing coat =
   UDL = -1.43 kN/m

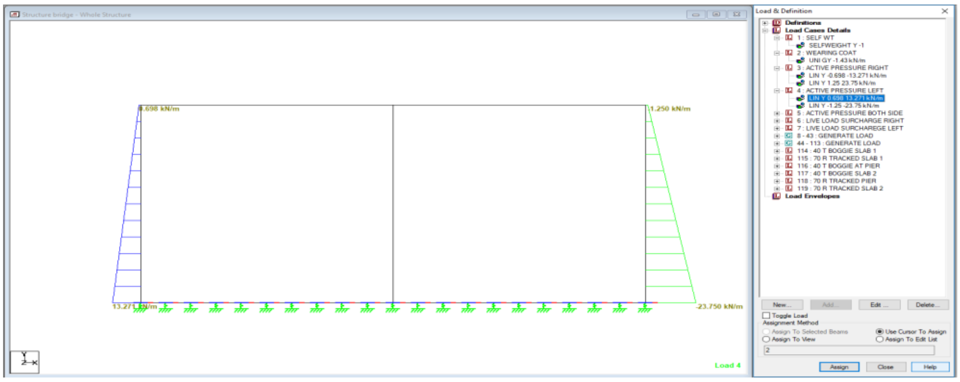
Active Pressure Right =

a. Linear Varying :- w1 = -0.698 kN/m
   w2 = -13.271 kN/m
b. Linear Varying :- w1 = 1.25 kN/m
   w2 = 23.75kN/m



Active Pressure Left =

a. Linear Varying: - w1 = 0.698 kN/m
   w2 = 13.271 kN/m
b. Linear Varying: - w1 = -1.25 kN/m
   w2 = -23.75kN/m

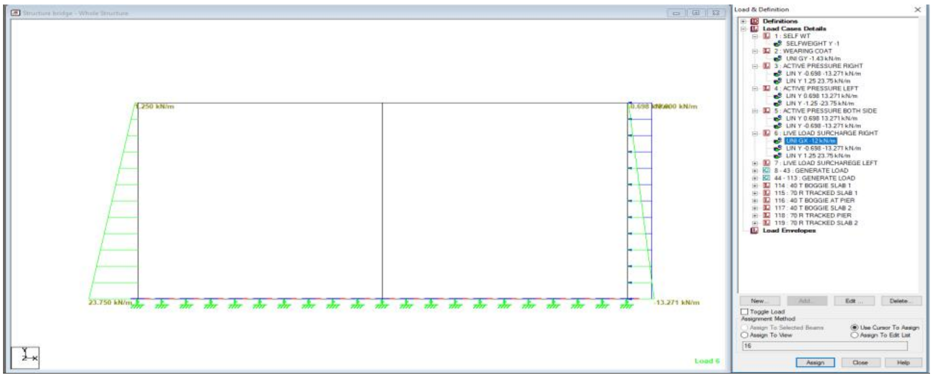


Active Pressure Both Side =

a. Linear Varying: - w1 = 0.698 kN/m
   w2 = 13.271 kN/m
b. Linear Varying: - w1 = -0.698kN/m
   w2 = -13.271kN/m

Live Load Surcharge Right =

a. Uniform force: - w1 = -12 kN/m
b. Linear Varying: - w1 = -0.698 kN/m
   w2 = -13.271 kN/m
c. Linear Varying: - w1 = 1.25kN/m
   w2 = 23.75 kN/m



Live Load Surcharge Left =
a. Uniform force: - w1 = 12 kN/m
b. Linear Varying:- w1 = 0.698 kN/m
w2 =13.271 kN/m
c. Linear Varying :- w1 = -1.25 kN/m
w2 = -23.75 kN/m

5.2 Live load

Considering two types of live load:

a. 40 tonboggie
b. 70R track

Critical Position of Live Load:
Summary of live load analysis has been done to know the position of load for maximum sagging moment at mid span and maximum hogging above the abutment and pier.

Table 2. Load analysis

Type of Loading	**No. of load case**	**Load acting**	**Distance**
40T Boggie	59	Wheel movement	0.1m
70R Track	93	span	4.50m
Location	Load type	Load case	Nature of bending
Slab 1	40T Boggie	12	Sagging
Slab 1	70R Track	26	Sagging
Above Pier	40T Boggie	41	Hogging
Above Pier	70R Track	48	Hogging
Slab 2	40T Boggie	47	Sagging
Slab 2	70R Track	67	Sagging

6 LOAD INTENSITY CALCULATION

For 70R Track: track load above slab 1, Impact factor = 25%

Table 3. Load calculations track load above slab 1

	LOADS	a(m)	b1(m)	beff (m)	beff/2 (m)	Next Wheel Load	Dispersion width	Load Intensity	unit
Start of UDL	437.5	0.00	0.97	0.970	0.485	2.060	0.970	98.694	kN/m
Mid of Span	437.5	1.125	0.97	2.433	1.216	2.060	2.246	42.619	kN/m
End of UDL	437.5	0.00	0.97	0.970	0.485	2.060	0.970	98.694	kN/m
	437.5	0.35	0.97	1.738	0.869	2.060	1.738	55.068	kN/m

For 70 R Track load above side wall location: Impact factor = 25%

Table 4. Load calculations above side wall location

	LOADS	a(m)	b1 (m)	beff (m)	beff/2 (m)	Next Wheel Load	Dispersion width	Load Intensity	unit
Start of UDL	437.5	0.350	0.97	1.738	0.869	2.060	1.738	55.068	kN/m
	437.5	1.125	0.97	2.433	1.216	2.060	2.246	42.619	kN/m
	437.5	0.00	0.97	0.970	0.485	2.060	0.970	98.694	kN/m
	437.5	1.125	0.97	2.433	1.216	2.060	2.246	42.619	kN/m
End of UDL	437.5	0.00	0.97	0.970	0.485	2.060	0.970	98.694	kN/m

For 70R Track: track load above slab 2, Impact factor = 25%

Table 5. Load calculations above slab 2

	LOADS	a(m)	b1 (m)	beff (m)	beff/2 (m)	Next Wheel Load	Dispersion width	Load Intensity	unit
Start of UDL	437.5	0.120	0.97	1.265	0.633	2.060	1.265	75.657	kN/m
	437.5	0.00	0.97	0.97	0.485	2.060	0.970	98.694	kN/m
	437.5	1.125	0.97	2.433	1.216	2.060	2.246	42.619	kN/m
End of UDL	437.5	0.00	0.97	0.97	0.485	2.060	0.970	98.694	kN/m

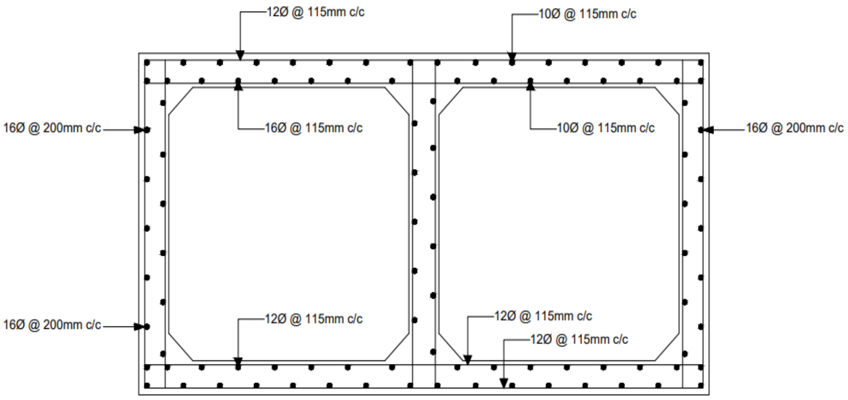


Figure 5. Reinforment details

Above diagram shows the reinforcement obtained from manual and computational approach. The detailing was found to be similar for both cases and hence validate the results.

7 RESULTS AND DISCUSSION

Calculations were done using manual approach and computational approach and Results were compared in the table below. Comparison of manual and Staad Pro results is shown in table Below. It is seen that results obtain from Staad Pro is much higher than that of manual approach. This is due to the fact that Staad Pro keeps much higher factor of safety than prescribed by the code in order to ensure that the structure is safe. Disparity in Bending Moment for Top slab may be because of different method (WSM and LSM) adopted for design.

Comparison Between Staad Pro and MDM Results

For 70R Track: (STADD PRO values)

Load above slab 1:

Position	STADD Values (KN-m)	MDM Values (KN-m)
Start of UDL	98.694	58.74
Mid of Span	42.619	28.3
End of UDL	98.694	58.74

Load above side wall location:

Position	STADD Values (KN-m)	MDM Values (KN-m)
Start of UDL	55.068	30.26
Mid of Span	20.44	10.15
End of UDL	98.694	52.55

Load above slab 2:

Position	STADD Values (KN-m)	MDM Values (KN-m)
Start of UDL	75.657	42.36
Mid of Span	70.65	38.25
End of UDL	98.694	58.74

8 CONCLUSION

The maximum design forces develop at these critical sections due to various combinations of loading patterns. The maximum design forces developed for the loading condition when the top slab is subjected to the dead load and live load and sidewall is subjected to earth pressure and surcharges, and when the culvert is empty. Finally concluded that Computational method (Staad Pro) was much more competent than Moment Distribution Method (MDM) in term of efficiency of result and time consumption.

CONFLICT OF INTEREST

The authors confirm that there is no conflict of interest to declare for this publication.

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