



PARAMETRIC STUDY ON RCC BOX FOR RAILWAY UNDER BRIDGE (RUB) CONSIDERING CUSHION THICKNESS AND SKEW ANGLE CONNECTION WITH DELHI RAILWAY LINE TO NAGPUR

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ABSTRACT

Abstract : Railway Under Bridges (RUBs) constructed using RCC box culverts are widely used for safe and efficient transportation. The structural behavior of these culverts is significantly influenced by factors such as cushion thickness and skew angle. This study aims to investigate the effect of varying cushion thickness and skew angle. This study aims to investigate the effect of varying cushion thickness and skew angle on the performance of RCC box culverts. In this research, a standard RCC box culvert model is analyzed using finite element -based software under different loading conditions, including dead load, live load and earth pressure. Cushion thickness of 0.5m, 1.0m, 1.5m and 2.0m along with skew angles of 0-degree, 15 degree, 30degree and 45 degrees are considered. Key parameter such as bending moment, shear force and deflection are evaluated for each case. The results indicates that an increase in cushion thickness reduce stress and bending moments, while higher skew angles lead to increased stress concentration and torsional effects. The study concludes that proper selection of cushion thickness and skew angle is essential for safe and economical design of RCC Box culverts,

Keyword: STAAD, Railway Under Bridge (RUB), RCC Box, Skew, cushion height
Civil Engineering, Design Study.

CHAPTER 1 INTRODUCTION

1.1. Overview

Railway Under Bridge are critical infrastructure element s that facilitate uninterrupted movement of road traffic beneath railway tracks. RCC box culverts are commonly adopted for RUB construction due to their structural strength, durability and ease of installation. However, with increasing traffic loads and environmental concerns, there is a growing need to evaluate and improve the design and performance of this structure.

This study focuses on analyzing the structural behavior of RCC box culverts by considering key influencing parameter such as cushion thickness and skew angle. Additionally, the study highlights the role of sustainable materials in enhancing durability and reducing environment impact. The research aims to provide insights for safer, economical and sustainable design of railway under bridge.

1.2. Introduction

RCC box culverts are widely used in the construction of Railway Under Bridges due to their high load carrying capacity and structural efficiency. These structures are subjected to various loads such as dead load, live load from trains, and earth pressure. The performance of RCC box culverts is significantly affected by parameter like cushion thickness and skew angle.



Figure 1.1: Image Showing RUB & ROB

Cushion thickness influences load distribution from the railway track to the culvert, while skew angle affects stress distribution from the railway track to the culvert, while skew angle affects stress distribution due to angular alignment. Despite several studies on box culvert design, limited research is available on the combined influence of these parameters along with sustainable material consideration.

This study aims to evaluate the structural behavior of RCC Box culvert under varying cushion thickness and skew angles, while also emphasizing the importance of sustainable construction practices.

1.3 Importance of Sustainable Materials

The use of sustainable materials in civil engineering has gained significant importance due to increasing environmental concern and the need for resources conservation. In the construction of RCC box culverts, traditional materials can be partially replaced or enhanced with sustainable alternatives such as fly ash, recycled aggregates, and supplementary cementitious materials.

These materials help in reducing carbon emissions, improving durability and minimizing the overall environmental impact of construction. For example, the use of fly in concrete improves workability and long-term strength, while recycled aggregates reduce the demand for natural resources.

1.4 Significance of the Study

This study is significant as it addresses key factors affecting the performance of RCC box culverts used in railway under bridges. By analyzing the effects of cushion thickness and skew angle, the research provides a better understanding of structural behavior under varying conditions.

The findings of this can help engineers in optimizing design parameters to achieve safe and economical structures. Additionally, the inclusion of sustainable material considerations promotes environmentally responsible construction practices.

1.5 Statement of the Problem

RCC box culverts used in railway Under Bridges are subjected to complex loading conditions, including dead load, live load from trains, and earth pressure. The structural performance of these culverts is significantly influenced by parameters such as cushion thickness and skew angle. However, in many conventional designs, these parameters are either simplified or not thoroughly analyzed.

Inadequate consideration of cushion thickness may lead to improper load distribution, while skew angle introduced additional stresses and torsional effects that are often neglected in basic design approaches. This can result in inefficient or unsafe structural performance.

1.6 Scope and Limitations

This study focuses on the structural analysis of RCC box culverts used in railway under bridges by Considering the effects of cushion thickness and skew angle. The analysis is carried out using finite element-based software under various loading conditions.

The study includes the evaluation of key structural parameters such as bending moment, shear force and deflection for different combinations of cushion thickness and skew angles. The results are compared to identify trends and optimize design parameters. Present study has certain limitation that should be considered while interpreting the results. The analysis is based on assumed geometric dimensions and material properties, which may vary in actual field conditions. The soil structure interaction is simplified and advanced nonlinear soil behavior is not fully considered. The study is limited to a specific range of cushion thickness and skew angles and extreme conditions are not analyzed.

CHAPTER 2

REVIEW OF LITERATURE

Sr. No.	Name and Details of paper and author	Important Findings	Relevance with objectives
1.	Agrawal and Singh (2023) parametric study on skew Box girder bridges	• Increase in skew angle leads to significant changes in torsion, shear forces and bending moment; effects become prominent beyond 30degree.	• Provides fundamental understanding of skew angle influence for parametric analysis.
2.	Agarwal et al. (2020) Alalysis of skew – curved RC box Girder Bridge	• Skewness causes non-uniform load distribution; higher stress concentration observed at acute corners.	Help in understanding stress distribution in skew RCC box structures.
3.	Kumar and Ingle (2015), Analysis of skew underpass Bridge using sap 2000	• Finite Element Method shows significant variation in internal forces compared to straight bridges.	• Support the use of FEM based parametric study in the present work.
4.	Recent RUB STUDY (2022) Effect of skew Angle and soil fill on Railway Underpass	• Increase in cushion thickness improves load dispersion; skew effect is more critical at lower cushion thickness.	• Directly relevant as it combines skew thickness effects.

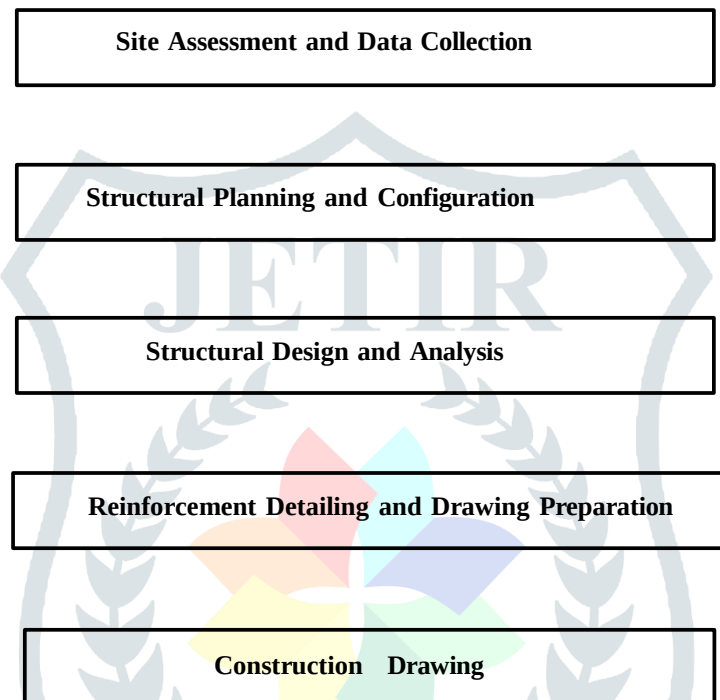
5.	IRC/IRS Codes Based Studies. Design of RCC Box Structures	Highlights importance of soil structures interaction and live load distribution in design.	Useful for validation of analytical results with design standards.
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Sr. No.	Name and Details of paper and author	Important Findings	Relevance with objectives
6.	FEM Based bridge studies (various) 3d finite element analysis of Box structures.	3d analysis provides more accurate results than 2d methods for box structures,	Justifies the methodology adopted in the present study.
7.	Soil Structure Interaction studies, Effect of soil cover on Underground structures	Higher cushion thickness resulted in better stress distribution and reduced live load impact.	Support inclusion of cushion thickness as a key parameter.
8.	Skew Bridge Behaviors Studies, Structural behaviors of skew bridges.	Skew bridges experience increased torsion and rotational effects due to inclined geometry.	Provides theoretical background for skew angle variation.
9.	Shimol Philipa, Rakendu R, Rajesh Lal. (December 2021) "The impact of geofom on the bending moment characteristics of reinforced concrete box culverts for road under bridge (RUB)design."	The main objective of this research is to investigate the effects of top-filling material properties on the bending moment characteristics of a two- cel IRC skew box culvert for RUB design.	The impact of geofom on the bending moment characteristics of reinforced concrete box culverts for road under bridge design.

CHAPTER 3 METHODOLOGY

3.1 Overview

The present study aims to investigate the structural behavior of an RCC box railway under bridge through a systematic parametric analysis considering the effects of cushion thickness and skew angle. The methodology is structured to simulate realistic field conditions and evaluate the response of the structure under varying parameters.



Flowchart 3.1: Flowchart of Methodology

3.1 Site Assessment and Data Collection

The study begins with the collection of relevant data required for the analysis and design of the RCC box railway under bridge. This includes: site details such as soil conditions, ground profile and embankment height. Railway loading data as per IRC 6. Material properties for concrete and steel

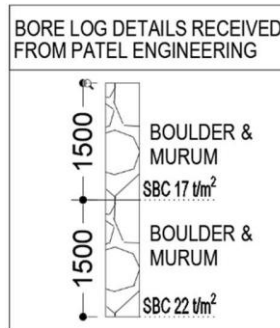
3.1.1 Topographical Data

Topographical data is collected to understand the physical characteristics of the site and to ensure accurate representation of field condition in the analysis. The data includes: Ground surface profile and natural slope of the terrain. Existing ground levels and reduced levels. Alignment of railway track and crossing angle. embankment height and formation width, location of existing drainage paths, nallahs or waterways, topographical information is obtained from survey data, contour maps or standard project report.

3.1.2 Soil Investigation Report

A detailed soil investigation is carried out to evaluate the sub surface condition at the proposed site of the RCC box railway under bridge, the Investigation provides essential geotechnical parameters required for analysis and design. The soil data is obtained from borehole investigation and in situ testing methods such as the standard Penetration test, along with laboratory testing of collected soil samples. The following parameters are determined from the soil investigation:

- Soil stratification and thickness of different layers
- Unit weight of soil
- Angle of internal friction and cohesion
- Safe bearing capacity of soil
- Groundwater table level



BORE LOG
SCALE - 1:100

3.1.3 Rail Alignment

The horizontal and vertical alignments for both road and railway were collected from the concerned authorities and verified through field observations.

The alignment data included: -

- Chainages of existing and proposed rail lines.
- Gradient and curvature details.
- Existing right-of-way (ROW) and clearance requirements.

These details were crucial for planning the layout of the underpass and ensuring compliance with Indian Railway standards.

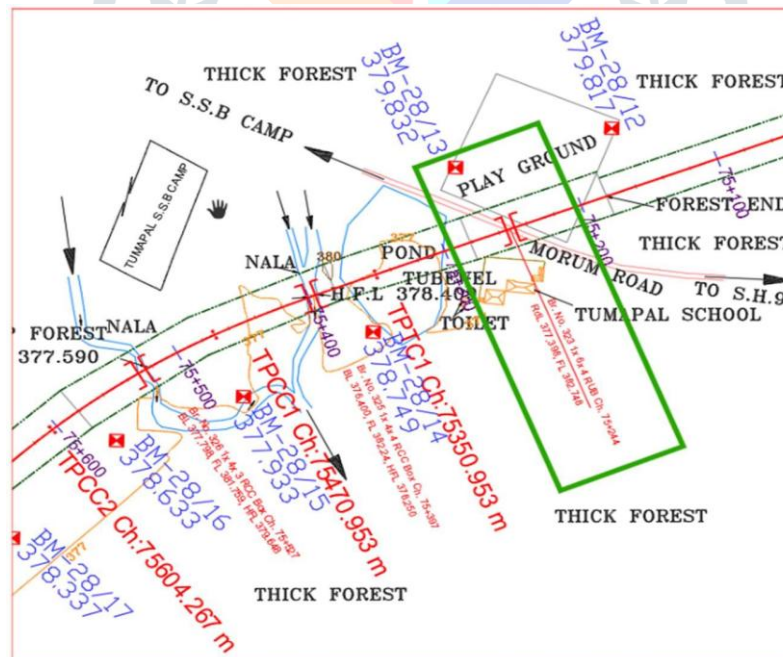


Figure 1.3 Site plan

3.1.4 Traffic Classification

Traffic surveys were conducted to classify and quantify vehicular movement on the existing road. The traffic data was categorized into light, medium, and heavy vehicles based on axle loads and frequency.

This classification was used for: -

- Estimating design traffic load.
- Determining the serviceability of the structure.

- Planning for future traffic growth and expansion provisions.

The data also helped in deciding the clear span, headroom, and number of lanes for the underpass.

- Verified skew angle based on the road and rail intersection and incorporated it into the design layout.
- Analyzed space constraints and clearance requirements as per Indian Railways guidelines.

3.2 Structural Planning and Configuration

The structural planning phase involves the conceptualization and arrangement of structural elements based on functional requirements, site constraints, and safety considerations. This stage is critical to ensure that the proposed structure meets the intended purpose, adheres to design standards, and remains economical and sustainable.

3.2.1 Objective of Structural Planning

The primary objective of structural planning is to develop a safe, serviceable, and durable structure that can effectively carry the imposed loads while satisfying aesthetic, operational, and maintenance requirements.

The planning was guided by: -

- Site-specific constraints
- Applicable design codes (e.g., IRS, IS:456, IRC:6, etc.)
- Clearance requirements for railway and roadway
- Traffic volume and future growth
- Soil and hydrological conditions

3.2.2 Type of Structure Selected

Based on the site investigation and functional needs, a Reinforced Cement Concrete (RCC) Box/Underpass/RUB (mention as applicable) was selected.

The selection was influenced by: -

- Ease and speed of construction
- Economical use of materials
- Suitability for low vertical clearance
- Compatibility with existing rail/road geometry

3.2.3 Structural Configuration

The selected RCC box structure comprises the following key components:

- Top Slab: Functions as the roof slab, designed to carry overburden, rail loads, and impact factors.
- Side Walls: Designed as vertical cantilevers or fixed-end elements to resist lateral earth pressure, surcharge, and live load effects.
- Bottom Slab: Acts as a base slab, resisting uplift pressure and providing stability.
- Wing Walls: Provided to retain backfill at the approaches and to guide traffic.

3.2.4 Typical dimensions: -

- Clear Span: 6.0 m
- Vertical Clearance: 4.0 m (as per RDSO/IRC specifications)
- Overall Width: 7.4 m (based on number of lanes)

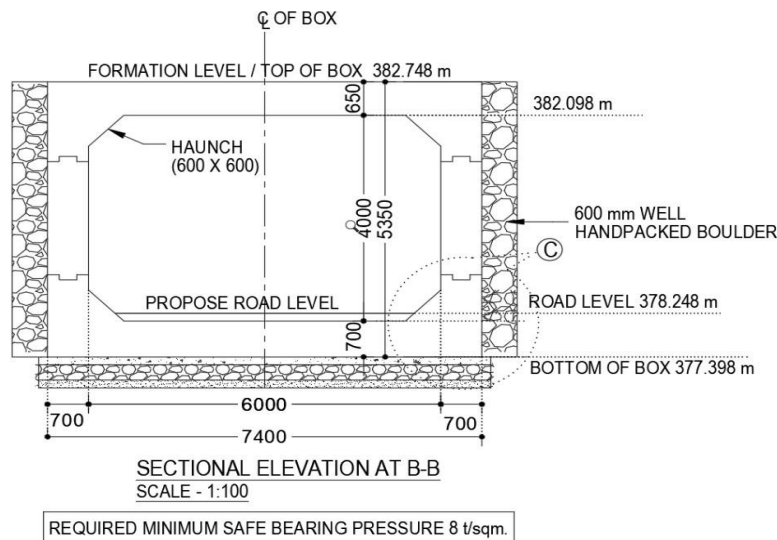
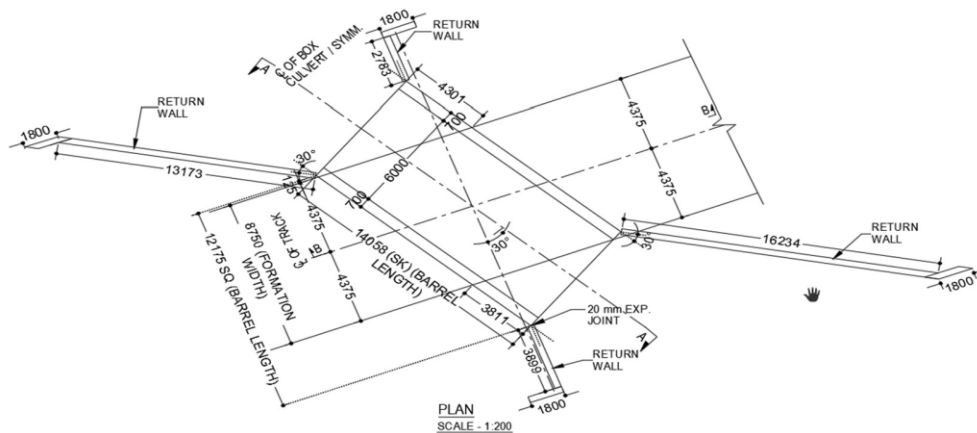


Figure 1.4: Dimensional Drawing



3.2.5 Structural Layout Considerations

Key considerations in layout planning included:

- Satisfying minimum clearance for road traffic.
- Avoiding existing utility lines and nearby structures.
- Ensuring effective drainage through slope and weep holes.
- Facilitating construction through phased execution.

3.2.6 Load Considerations

Designing a civil engineering structure involves the comprehensive understanding and application of various loads that the structure will be subjected to during its service life. For the structure under consideration (e.g., RCC box culvert, underpass, or retaining wall), loads are classified broadly into permanent (dead) and variable (live or transient) loads, including environmental and special loads. The design must comply with relevant Indian standards such as IS: 456-2000, IS: 875 (Part 1 to 5), IRC:6, IRS bridge rules, and IS:1893 for seismic loading.

• **Dead Loads**

These are permanent loads that do not change over time. They include:

- **Self-weight of the structure:** This includes the weight of concrete, reinforcement, and any other embedded materials. It is computed using the unit weight of materials as per IS:875 (Part 1). For concrete, the typical density is taken as 25 kN/m³.
- **Superimposed dead loads:** These include loads such as ballast in railway bridges, pavement layers in road bridges, wearing coats, parapets, and utilities (drainage pipes, cables, etc.).

Dead loads are calculated based on geometric dimensions and material densities and are always acting vertically downward.

• **Live Loads**

Live loads are dynamic or variable loads due to usage:

- **For road structures:** Live loads are considered as per IRC:6, adopting vehicle loading such as:
- IRC Class A Loading: Standard loading for typical highways.
- IRC Class AA Loading: For bridges on important roads or highways, subjected to heavy traffic or defense requirements.
- **For railway structures:** As per IRS Bridge Rules, railway loading includes:
- Class A, B, 70R Loading (BG, MG, or NG)
- Load Model for electric or diesel locomotives
- Impact factors are applied to account for dynamic effects.

Live loads are applied in various configurations to create the worst-case scenario for bending, shear, and bearing pressure.

• **Earth Pressure**

When designing retaining structures or underground RCC boxes, lateral earth pressure must be carefully evaluated. This pressure depends on:

- Type of soil
- Soil backfill height
- Water table location
- Wall movement (active/passive condition) The two primary theories used are:
- Rankine Theory: Assumes a frictionless wall and is simpler. Used where backfill is level and the wall is smooth.
- Coulomb Theory: Considers wall friction and inclined backfill. Suitable for complex geometries.

Earth pressure is applied as triangular or trapezoidal distribution and calculated using:

- Active Pressure (Pa): When wall moves away from soil.
- Passive Pressure (Pp): When wall resists movement into the soil.

• **Surcharge Loads**

Surcharge is an additional vertical load acting on the backfill above the structure due to nearby traffic, rail tracks, or structures. This surcharge induces additional lateral earth pressure.

- Uniform surcharge (q): Induces additional lateral pressure.
- Strip surcharge (from nearby roads or rail): More accurate methods (e.g., Boussinesq or equivalent line load theory).

• **Seismic and Wind Loads**

Seismic Loads: Seismic forces are significant in earthquake-prone zones. As per IS:1893 (Part 1):

- Structures are designed for lateral forces due to ground acceleration.
- Seismic coefficient method or response spectrum method is used.
- Seismic earth pressure is higher than static pressure.

Seismic forces act horizontally and increase the demand on retaining walls and underground structures.

Wind Loads:

Generally, for underground structures or those close to ground, wind loads are negligible. However, for elevated parts like parapets or noise barriers, wind loads may be considered using IS:875 (Part 3).

Each load has a unique effect on the structure and must be carefully combined using load combinations prescribed in IS:456 and IRC/IRS codes. The critical load combinations are then used for ultimate limit state (ULS) and serviceability limit state (SLS) checks. Proper understanding and application of these loads ensure safety, serviceability, and durability of the structure.

The structure was planned to resist various loads in accordance with Indian Standards:

- Dead loads (self-weight, superimposed loads)
- Live loads (traffic loads conforming to IRC Class A, AA, or railway loading as per IRS)
- Earth pressure (calculated using Rankine or Coulomb theory)
- Surcharge loads due to adjacent road or rail.

3.2.7 Durability and Service Life

Durability and service life are critical aspects of structural design, particularly for infrastructure elements exposed to harsh environmental conditions, such as railway underpasses, culverts, retaining walls, and bridges. A durable structure is one that can withstand environmental actions, chemical exposure, mechanical wear, and loading effects without significant loss of serviceability or structural integrity over its intended lifespan.

As per IS:456-2000 and relevant IRC/IRS codes, achieving durability requires a holistic approach—starting from material selection, structural detailing, exposure classification, and maintenance strategies.

• Definition of Durability

Durability refers to the ability of a structure to withstand the effects of weathering, chemical attack, abrasion, or any other process of deterioration, while maintaining its strength and serviceability.

Factors influencing durability include:

- Quality of concrete (grade, water-cement ratio, compaction, curing)
- Type and quality of reinforcement
- Exposure conditions (aggressive soil, water, chemical exposure, freeze-thaw)
- Environmental factors (temperature, humidity, rainfall, pollutants)
- Maintenance and inspection frequency

• Service Life of Structure

Service life is defined as the period during which the structure performs its intended function without requiring major repair or rehabilitation.

As per IRC SP:96 and international practices (e.g., fib Model Code), the service life of infrastructure structures is typically classified as:

Structure Type	Expected Service Life
Minor Bridges/Culverts	50 years
Major Bridges/RUBs	100 years
less	Temporary Works 25 years or less

Service life is influenced by the environment, design detailing, load spectrum, and maintenance regimes.

The following standards provide guidelines for achieving durability:

- IS:456-2000 – General guidelines for concrete structures
- IRC:112-2020 – Code of practice for concrete road bridges
- IRC: SP-96 – Guidelines for service life design of highway bridges
- IRS Concrete Bridge Code – For railway structures

3.3 Structural Design and Analysis

The structural design and analysis of the proposed RCC box structure were carried out using advanced computational tools and in accordance with the latest Indian Standard Codes. The objective of the analysis was to ensure that the structure is safe, durable, and serviceable under all critical loading conditions.

3.3.1 Modeling and Load Application

The structure was modeled using STAAD.Pro, a widely accepted finite element analysis software, to simulate realistic boundary conditions, loading scenarios, and material behavior. The RCC box was modeled as a 3D space frame using plate and beam elements to accurately capture the interaction between structural components.

The following loads were applied as per applicable codes:

- Dead Load (DL): Self-weight of the structure automatically calculated by STAAD based on member dimensions and material density (25 kN/m³ for concrete).

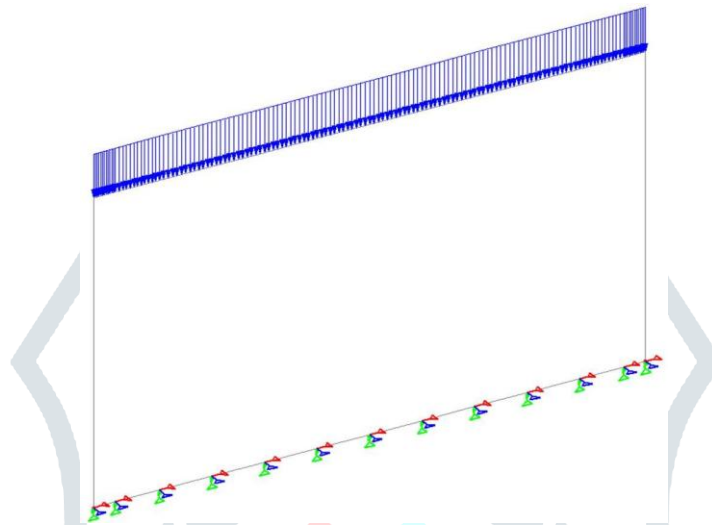


Figure 1.5: Dead load

- Live Load (LL):
 - For road-carrying structures, IRC Class A or Class 70R tracked loading was applied.
 - Live load distribution, impact factors, and load positioning were considered as per IRC:6.
- Earth Pressure:
 - Calculated using Rankine's Theory assuming active pressure condition.
 - Applied as lateral pressure on vertical side walls with a triangular distribution based on backfill height and soil properties.

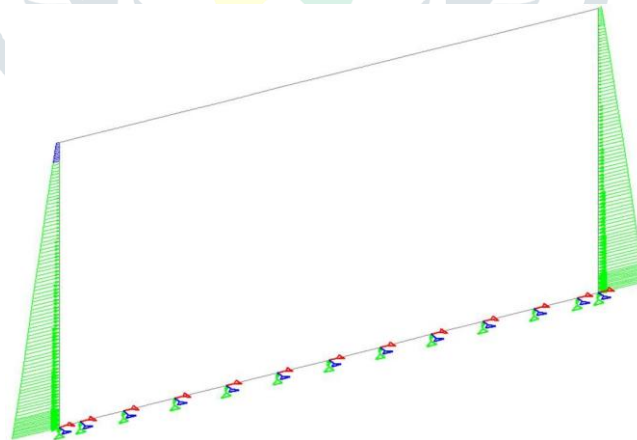


Figure 1.6: Active Earth Pressure

- Surcharge Load:
 - Representing traffic load adjacent to the wall or structure.
 - Modeled as a uniform load on the backfill surface, converted into equivalent lateral pressure using $q.K_a$.

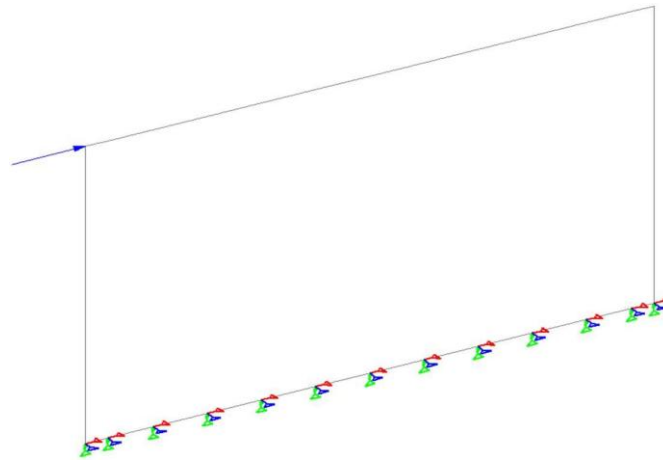


Figure 1.7: Breaking force

- Seismic Load:
- Considered as per IS:1893 (Part 1) and IRC:6 for seismic zone classification.
- Horizontal acceleration coefficients applied based on seismic zone factor, importance factor, and soil type.
- Wind Load (if applicable):
- Considered only for exposed elements or elevated sections, using IS:875 (Part 3).

3.3.2 Load Combinations and Limit States

Comprehensive load combinations were generated based on the guidelines of:

- IS:456-2000 – For concrete structures
- IRC:6-2017 – For highway bridges
- IRC:112-2020 – For limit state design of road bridges
- IRS Bridge Rules – For railway structures

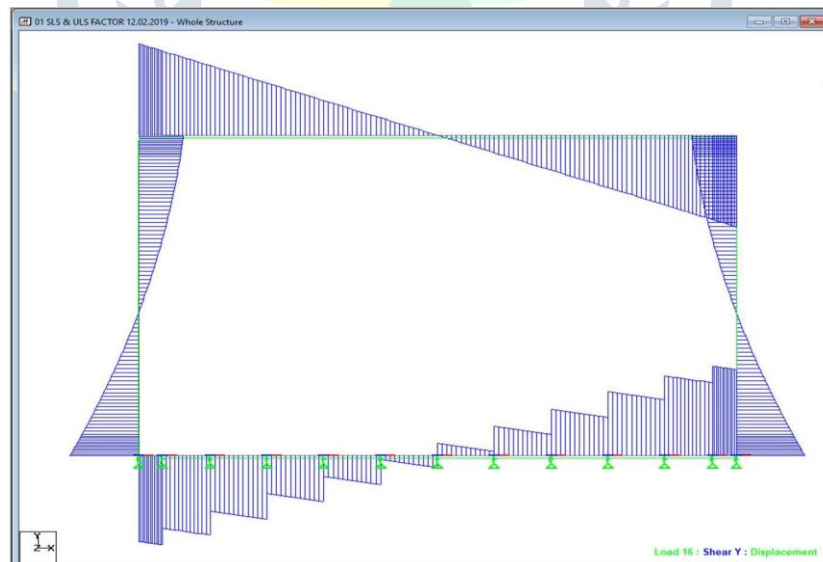


Figure 1.8: Shear Force Diagram



Figure 1.9: Bending Moment Diagram

Both Ultimate Limit State (ULS) and Serviceability Limit State (SLS) conditions were checked:

- ULS: Ensures safety against collapse (strength limit)
- SLS: Ensures control of deflection, cracking, and comfort during service life

Typical load combinations used:

- DL + LL + Earth Pressure
- DL + LL + Earth Pressure + Surcharge
- DL + LL + Earth Pressure + Seismic
- DL + LL + Uplift (for submerged conditions)

3.3.3 Structural Checks and Verifications

After running the analysis, the structure was checked for the following performance criteria:

- Bending Moment and Shear Force Capacity
 - The maximum bending moments and shear forces were obtained from the analysis.
 - Reinforcement was designed using limit state method to resist these values with adequate safety margins.
 - Section capacities were verified against design demands using interaction equations for combined loading.

- Earth Pressure on Side Walls
 - Lateral forces from active earth pressure were verified using Rankine theory:

$$P_a = 1/2 \cdot \gamma \cdot H^2 \cdot K_a$$

Where:

γ = unit weight of backfill H = height of wall

K_a = active earth pressure coefficient

- Reinforcement was provided to resist these lateral loads, ensuring no overturning or sliding.

- Structural Deflection and Crack Control
 - Maximum deflections were checked under service load combinations.
 - Deflection limits as per IS:456 and IRC:112 (typically span/250) were followed.
 - Crack width control was ensured by proper reinforcement detailing and spacing.
 - Maximum allowable crack width: 0.3 mm (moderate environment)
 - Spacing and bar diameter limits applied as per IRC:112 guidelines

3.4 Reinforcement Detailing and Drawing Preparation

In the present study, detailed reinforcement drawings were prepared for the structural components of the RCC box, including the top slab, bottom slab, side walls, and haunches. The reinforcement detailing was carried out in accordance with the provisions of IS:456-2000 (Code of Practice for Plain and Reinforced Concrete) and SP:34 (Handbook on Reinforcement and Detailing), ensuring structural safety and serviceability.

The drawings incorporated all essential reinforcement parameters such as bar diameters, spacing, concrete cover, anchorage lengths, and lap lengths, considering both flexural and shear reinforcement requirements. Special attention was given to critical zones like haunch regions and slab-wall junctions to ensure adequate structural performance under applied loads.

All reinforcement details and drawings were prepared using AutoCAD, following the standard drawing conventions and guidelines prescribed by the Indian Railways. This ensured uniformity, accuracy, and ease of interpretation by field engineers and construction personnel.

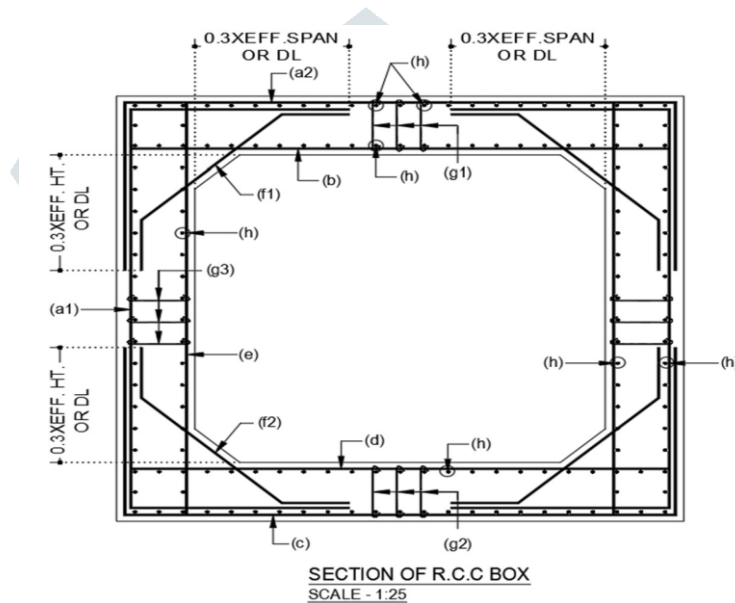


Figure 2.0: Reinforcement Detailing

Bar Marked	Dia (mm)	Spacing (mm)	Area cm ²	Remark
a1	25	200	24.53	Vertical wall outer
a2	25	200	24.53	Top bar of top slab
b	25	100	49.06	Bottom bar of top slab
c	25	200	24.53	Bottom bar of bottom slab
d	25	100	49.06	Top bar of Bottom slab
e	20	100	31.40	Inner bar of Wall
f1	25	200	24.53	Top haunch bar
f2	25	200	24.53	Bottom haunch bar
g1	10	200	3.93	Shear link for Slab
g2	10	200	3.93	Shear link for Raft
g3	10	200	3.93	Shear link for Wall
h	12	100	11.30	Distribution Outer wall
h	12	100	11.30	Distribution Bottom slab

Table 1.1 – Reinforcement Provided

3.4.1 Construction Drawing and Documentation

- Created construction-ready drawings, including plan, sections, elevations, and reinforcement layouts with notes

and dimensions.

- Added skew angle adjustments and labeled all sections and reinforcement placement clearly.
- Submitted a detailed design calculation report, technical notes, and drawing set to the approving authority.

3.4.2 Challenges Faced and Solutions

- **Challenge:** Handling skew angle in analysis and detailing

Solution: added diagonal reinforcement.

- **Challenge:** Detailing reinforcement in corner zones and joints

Solution: enhanced reinforcement overlaps and anchorage

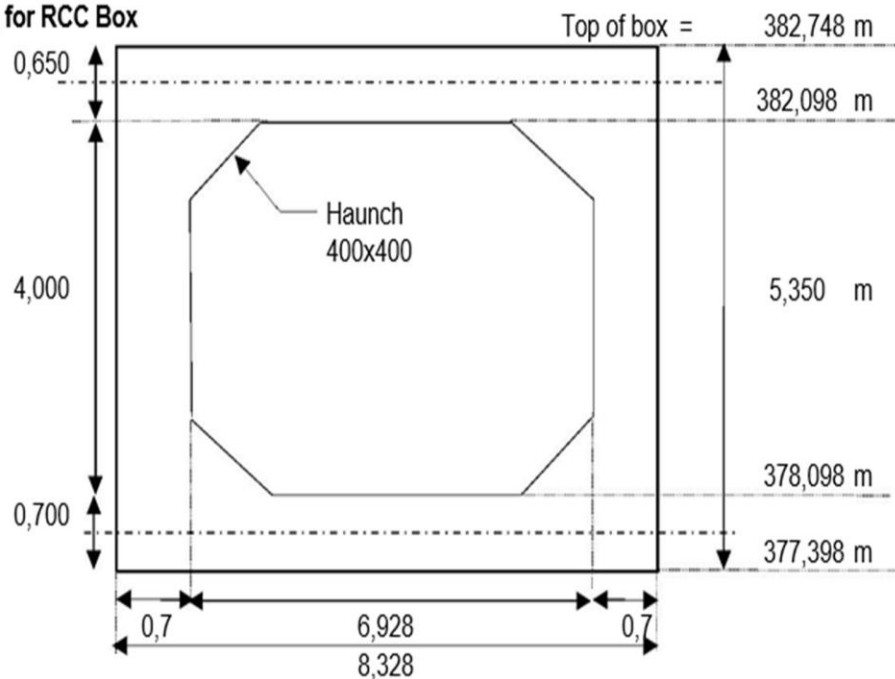
- **Challenge:** Time management for design, review, and drafting

Solution: Scheduled structured phases and review cycles for quality control

CHAPTER 4 CALCULATION

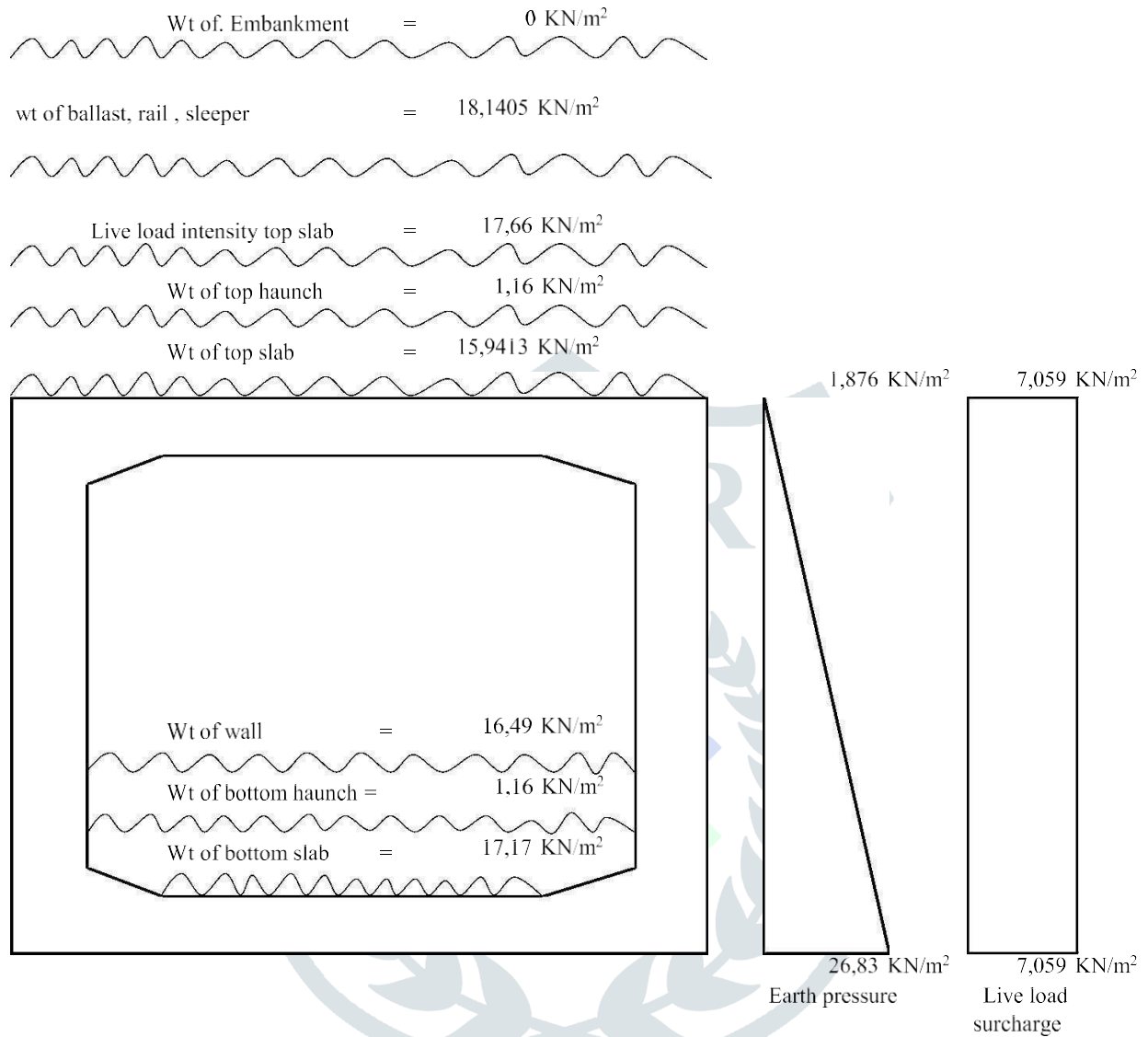
DESIGN OF R.C.C. BOX

1) Drawing for RCC Box



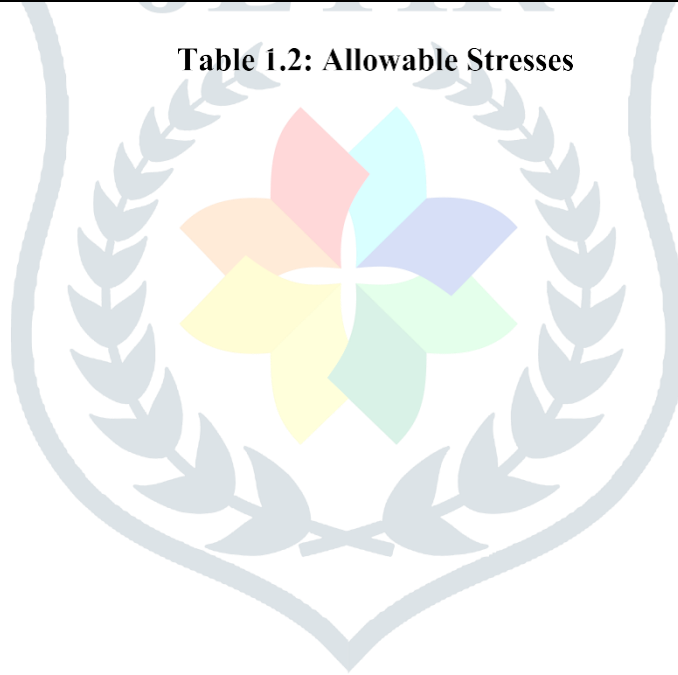
CROSS SECTION OF R.C.C. BOX

Total length of box = 8,328 m

Chapter4: Calculation**2) LOADING DIAGRAM**

*Chapter4: Calculation***3) Allowable Stresses IRC: 112-2011, A4.5.1, Table A4.2, page no. 253**

SN.	Allowable Stresses					
1)	Reinforcement (Normal Condition)					
	Steel Fe 500					
a)	Flexure Tension	=	240 N/mm ²	x	1,02	2446 Kg/cm ²
b)	Shear	=	200 N/mm ²	x	1,02	2039 Kg/cm ²
c)	Direct Compression	=	205 N/mm ²	x	1,02	2090 Kg/cm ²
2)	Concrete M45					
a)	Permissible direct compressive stresses	=	11,25 N/mm ²	x	1,02	115 Kg/cm ²
b)	Permissible flextural compressive stresses	=	15 N/mm ²	x	1,02	153 Kg/cm ²
3)	Modular Ratio	=	10			

Table 1.2: Allowable Stresses

Chapter4: Calculation

4) Data available

a) No. of boxes	=	1 Nos.
b) Clear span square	=	6,928 m
c) Height of box	=	5,350 m
d) Depth of deck slab	=	0,650 m
e) Depth of bottom slab	=	0,700 m
f) Thickness of side walls	=	0,700 m
h) Grade of concrete	=	M 35
i) Grade of steel	=	Fe500
j) Bulk density of soil	=	19 KN/m ³
k) Angle of internal friction	=	35
m) Unit wt. of concrete	=	2,50 t/m ³
n) Thickness of haunches	=	0,600 m
o) Size of haunch	=	0,600 m
p) Unit wt. of water	=	10
q) Skew angle	=	30,00 °
r) Live load surcharge	=	1,2 m Height
s) Clear span	=	6,928 m
t) Rail Level	=	383,486 m
u) Top of box level	=	382,748 m
v) Overburden soil	=	0,000 m

4.1) Load Considered.

- Dead load of slab
- Dead load of Walls
- Earth pressure due to back fill.
- Earth pressure due to Live load surcharge. 1,2 m ht
- Longitudinal Force due to Breaking force
- Water Pressure

4.2) Calculations: -

Effective span straight	=	7,628 m
Effective height	=	4,675 m
Total length of span at bottom	=	8,328 m
Total height of box	=	5,350 m
Depth of earth fill	=	0,000 m

The bending moment in slab are evaluated using computer program

The loading on subway is considered according to IRC.

The moment due to self weight is evaluated by considering unit weight of concrete as 2.5 t/m³ for all longitudinal member of the grid.

The earth pressure including live load surcharge is loaded on the the vertical members of the grid. The back fill behind the walls of the box shall be as per IRC

- Grade of concrete for the box M 35
- Grade of steel Fe500
- The permissible stresses as per IRC

- Minimum S.B.C. required is 7 t/m².

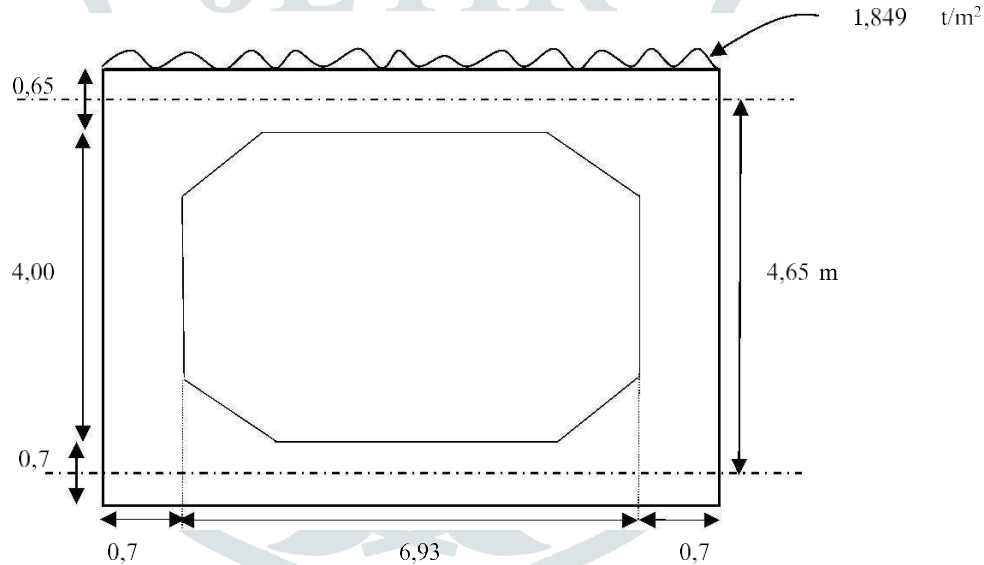
- Verify safe bearing capacity at site before execution.

Chapter4: Calculation

5) CALCULATION OF DEAD LOADS

5.1) SUPERIMPOSED DEAD LOAD FOR

i) Wt of concrete	=	2,5	t/m ³	
ii) Height of Embankment	=	0,000	m	
iii) Intensity of DL per square meter	=	1,9	x	0 = 0,000 t/m ²
iv) Wt. of Ballast	=	0,7	x	2,2 = 1,54 t/m ²
v) Wt. of Rail	=	0,062	x	2 = 0,124 t/m ²
vi) Wt. of Sleeper	=	0,300	x	$\frac{1,667}{2,7} = 0,185185$ t/m ²
vii) Total Super Imposed Load Intensity	=	0,000 + 1,54 + 0,12 + 0,19		
	=	1,85	t/m ²	



5.2) DEAD LOAD

a) Top Slab	=	0,65	m	
Self weight of Slab	=	0,65	x	2,5 = 1,625 t/m ²
Wt. of haunch	=	0,180	x	2,5 = 0,450 t/m
for 2 no of haunch	=	$\frac{0,450}{2}$	x	2 = 0,1180 t/m ²
Weight of top slab	=	1,625	+	0,1180 = 1,74 t/m ²

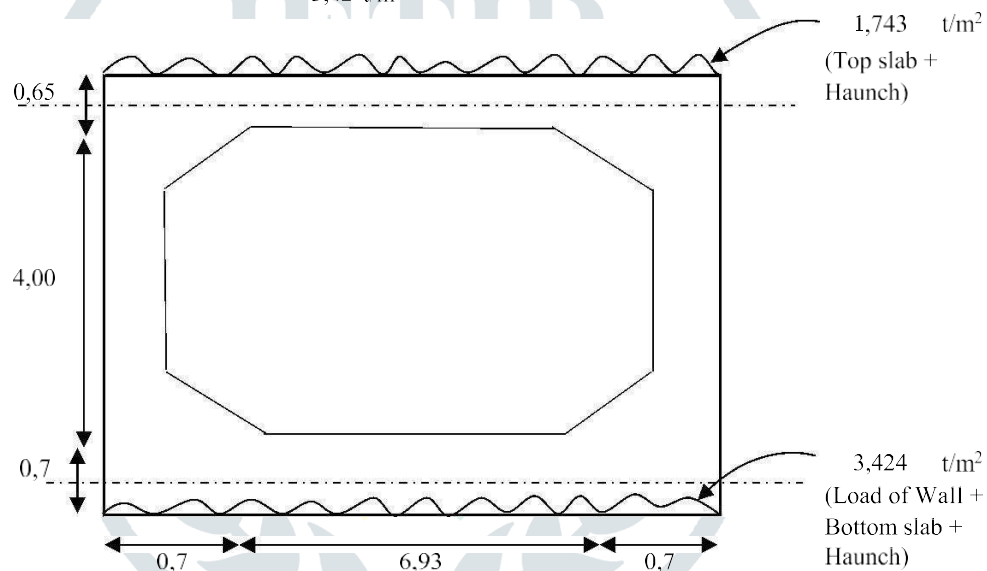
Chapter4: Calculation**5.3) WEIGHT OF WALL**

i) Wall Thickness	=	0,7 m			
ii) Wt of wall/ height	=	0,7	x	2,5	= 1,750 t/m ²
iii) Weight of bottom slab	=	0,7	x	2,5	= 1,750 t/m ²

iv) Due to weight of two wall UDL on bottom slab

$$\begin{aligned}
 \text{UDL} &= \frac{\text{Wt. of walls x No.}}{\text{Effective Span}} \\
 &= \left[\frac{1,75 \times 2}{8,328} \right] \times \text{Ht. of Wall} \\
 &= 1,681 \text{ t/m}^2
 \end{aligned}$$

$$\begin{aligned}
 \text{v) Total wt.on bottom slab} &= 1,625 + 1,681 + 0,12 \\
 &= 3,42 \text{ t/m}^2
 \end{aligned}$$



Chapter4: Calculation

6.3) Live Load for Rail

As per Bridge Rule, Appendix - XXIII (a), Page no. 83

For 6,928 m span = 99,09 t

It is equally distributed on 14,058 m width
and 6,928 m

$$= \frac{99,09}{14,058 \times 6,928} = 1,02 \text{ t/m}^2$$

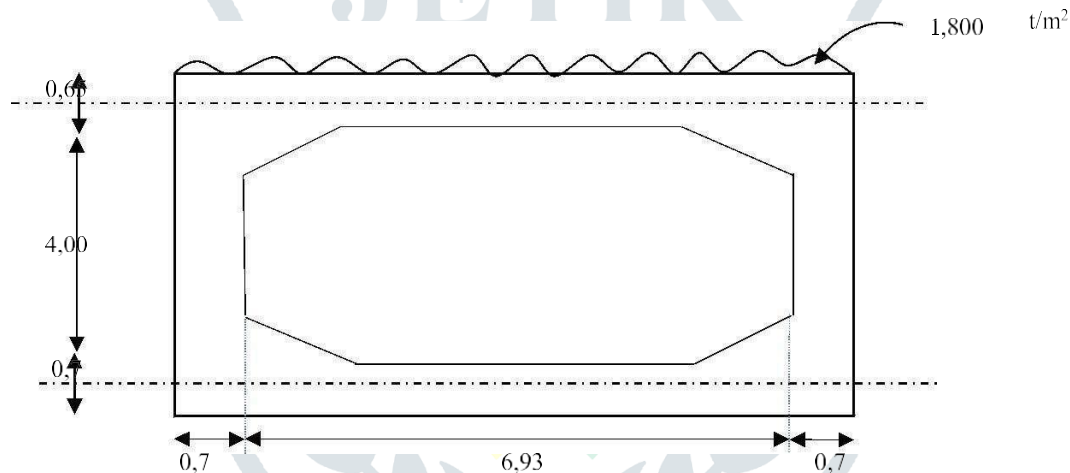
Impact

As per Bridge Rule, Appendix - XXIII, Page no. 81

For 7,628 m span = 0,769

Live Load with Impact

$$= 1,02 \times 1,769 = 1,800 \text{ t/m}^2 = 17,656 \text{ KN/m}^2$$



Chapter4: Calculation

5) CALCULATION OF EARTH PRESSURE (ACTIVE)

a) Coefficient of active earth pressure (As per IRC)

Angle of repose	Φ	35	Degree
Angle of wall friction	δ	10	Degree
Inclination earth face of wall	α	0	Degree
Slope of Back fill	i	0	Degree
Dry Density of Backfill		1,9	t/m ³
Density of water		1	t/m ³

$\Phi - \alpha$	=	25	$\cos (\Phi - \alpha)$	=	0,91
$\alpha + \delta$	=	10	$\cos (\alpha + \delta)$	=	0,98
$\Phi + \delta$	=	45	$\sin (\Phi + \delta)$	=	0,71
$\Phi - i$	=	35	$\sin (\Phi - i)$	=	0,57
$\alpha - i$	=	0	$\cos (\alpha - i)$	=	1,00
α	=	0	$\cos \alpha$	=	1,00
δ	=	10	$\cos \delta$	=	0,985

$$k_a = \frac{\cos^2(\Phi - \alpha)}{\cos^2 \alpha * \cos(\alpha + \delta) * (1 + ((\sin(\Phi + \delta) * \sin(\Phi - i)) / (\cos(\alpha + \delta) \cos(\alpha - i)))^{0.5}}$$

$$k_a = \frac{0,822}{1,000 \times 0,98 \times 2,694}$$

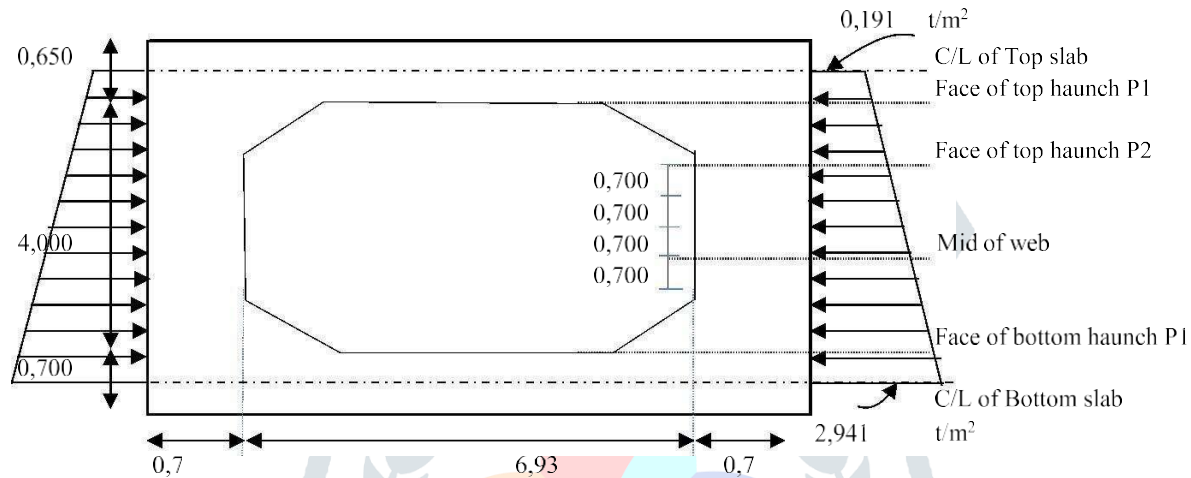
$$= 0,310$$

$$k_a = 0,310$$

Chapter4: Calculation

b) Active Earth pressure

i) Height of back fill	=	H	=	5,000	m
ii) Height of overburden	=	h	=	0,000	m



1) Earth pressure

Height (m)		$K_a \times \gamma \times H$			
5,000	at C/L of bottom slab	=	0,310	x 1,9	x 5,000 = 2,941 t/m²
4,65	at Face of bottom haunch P1	=	0,310	x 1,9	x 4,650 = 2,735 t/m²
4,05	at Face of bottom haunch P2	=	0,310	x 1,9	x 4,050 = 2,382 t/m²
3,350	at Web	=	0,310	x 1,9	x 3,350 = 1,971 t/m²
2,650	at Web	=	0,310	x 1,9	x 2,650 = 1,559 t/m²
1,950	at Web	=	0,310	x 1,9	x 1,950 = 1,147 t/m²
1,250	at Web	=	0,310	x 1,9	x 1,250 = 0,735 t/m²
0,650	at Face of top haunch P1	=	0,310	x 1,9	x 0,650 = 0,382 t/m²
0,325	at C/L of Top slab	=	0,310	x 1,9	x 0,325 = 0,191 t/m²

Chapter4: Calculation

5) CALCULATION OF EARTH PRESSURE (AT REST)

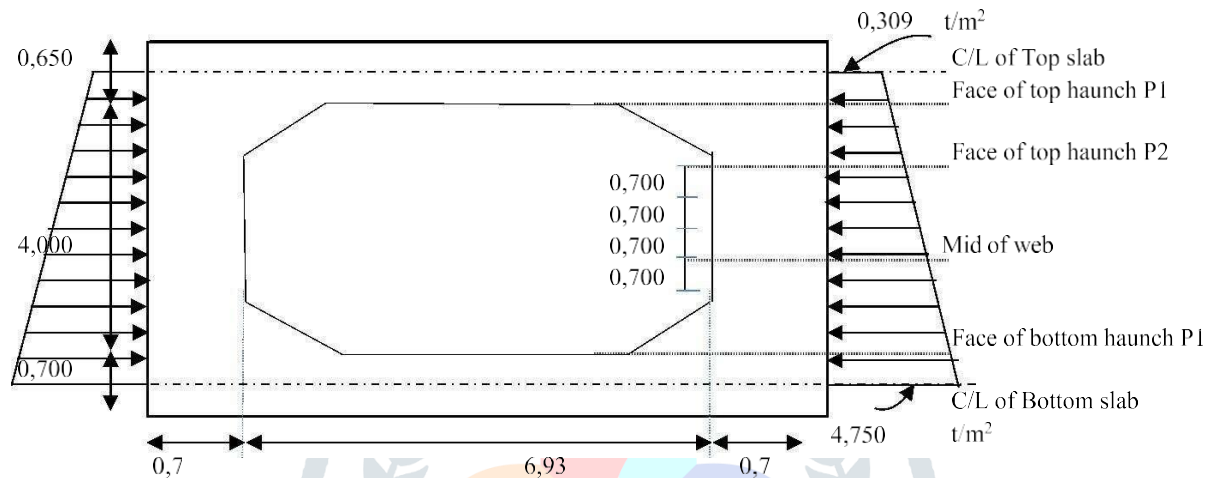
$$k_a = 0,310$$

$$k_a = 0,500$$

b) Active Earth pressure

i) Height of back fill = $H = 5,000$ m

ii) Height of overburden = $h = 0,000$ m



1) Earth pressure

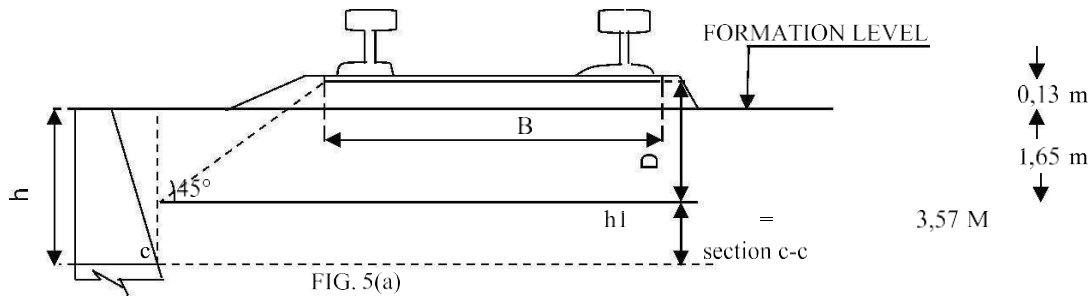
Height (m)		$K_a \times \gamma \times H$			
5,000	at C/L of bottom slab	=	0,500	x 1,9	x 5,000 = 4,750 t/m ²
4,65	at Face of bottom haunch P1	=	0,500	x 1,9	x 4,650 = 4,418 t/m ²
4,05	at Face of bottom haunch P2	=	0,500	x 1,9	x 4,050 = 3,848 t/m ²
3,350	at Web	=	0,500	x 1,9	x 3,350 = 3,183 t/m ²
2,650	at Web	=	0,500	x 1,9	x 2,650 = 2,518 t/m ²
1,950	at Web	=	0,500	x 1,9	x 1,950 = 1,853 t/m ²
1,250	at Web	=	0,500	x 1,9	x 1,250 = 1,188 t/m ²
0,650	at Face of top haunch P1	=	0,500	x 1,9	x 0,650 = 0,618 t/m ²
0,325	at C/L of Top slab	=	0,500	x 1,9	x 0,325 = 0,309 t/m ²

Chapter4: Calculation

9) LIVE LOAD SURCHARGE

Surcharge P1

$$\begin{aligned}
 L &= 7,85 \text{ m} \\
 B &= 3,152 \text{ m} \\
 h_1 &= \frac{L-B}{H} = \frac{4,698}{5,35} < 1,65 \\
 D &= 0,13 \text{ m} \\
 &= 1,78 \text{ m} \\
 h_1 &= 5,350 - 1,78 = 3,57 \text{ m}
 \end{aligned}$$

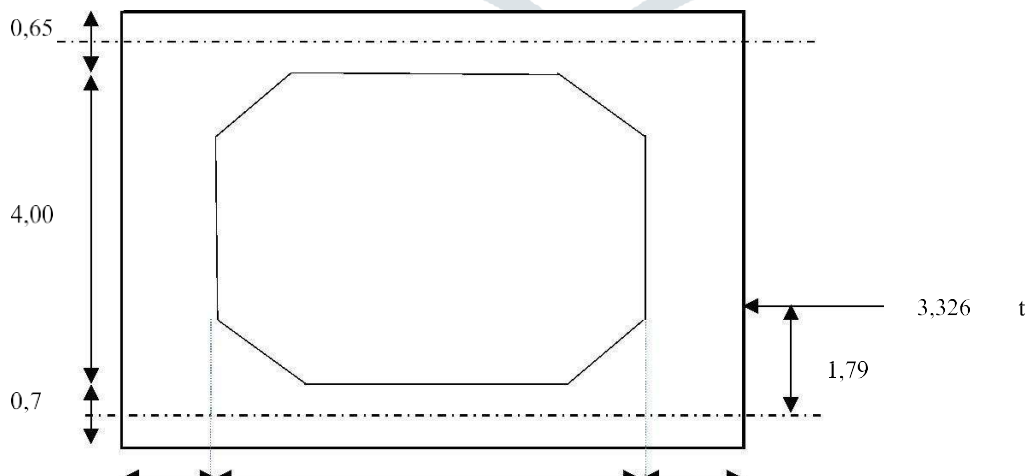


$$\begin{aligned}
 S &= \text{Live Load Surcharge Per Unit Length} \\
 &= 13,7 \text{ t/m}
 \end{aligned}$$

$$\begin{aligned}
 V &= \text{Dead Load Surcharge Per Unit Length} \\
 &= 6,5 \text{ t/m}
 \end{aligned}$$

$$\begin{aligned}
 P1 &= \frac{\left[\frac{S}{B} + \frac{V}{2D} \right] \times Ka \times h_1}{1} \\
 &= \frac{\left[\frac{13,7}{3,152} + \frac{6,5}{2 \times 0,310} \right] \times 1,78}{1}
 \end{aligned}$$

$$\begin{aligned}
 &= 3,33 \text{ t} \\
 \text{Acts at} &= \frac{h_1}{2} \\
 &= \frac{3,57}{2} \\
 &= 1,79 \text{ m}
 \end{aligned}$$



Chapter4: Calculation**10) LONGITUDINAL LOADS****10.1) Max Longitudinal Force**

In staad Model

applied on two points

This force is applied on two points

$$= 33,32 \text{ t}$$

$$= \frac{33,32}{2}$$

$$= 16,660 \text{ t (per 2 m)}$$

width of box

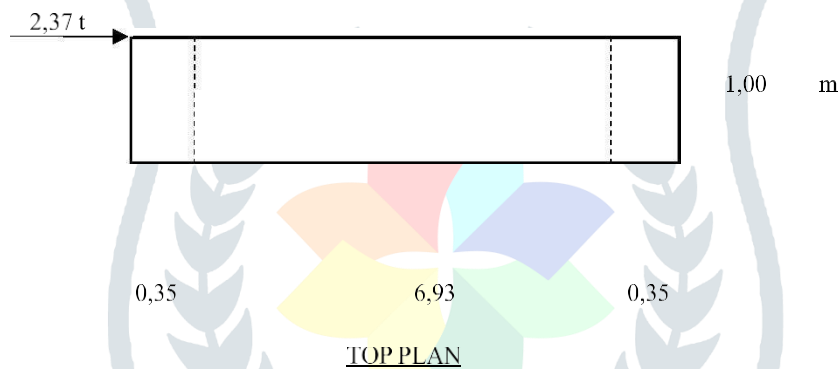
$$= 14,058 \text{ m}$$

Longitudinal Force

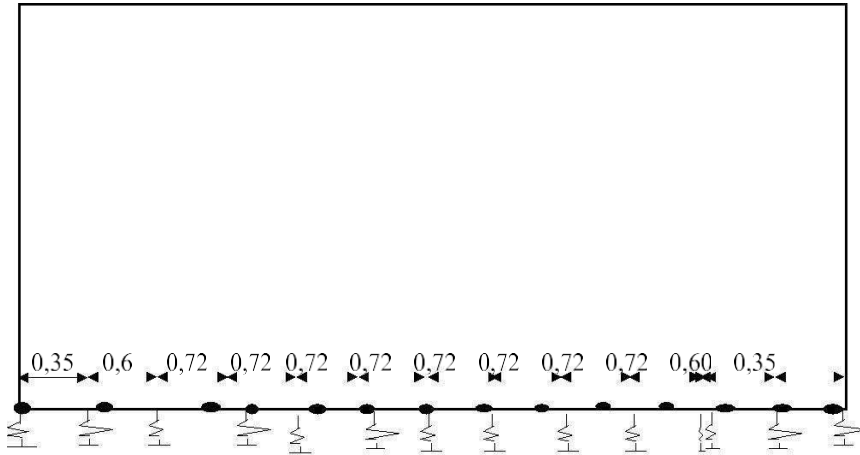
$$= 2,370 \text{ t (per m)}$$

force is applied on points

$$= 2,37 \text{ t (per point)}$$



12) Spring Constant



Allowable Safe Bearing Pressure (q_a) = 2 t/m²

Factor of Safety (S.F.) = 2,5

Modulus of Subgrade reaction
($K_s = 40 \times S.F. \times q_a$) = 243 t/m³

SN	Member Length (m)	L (m)	B (m)	KFY (KN/m ³)	KFY (KNm)	Joint
1	0,350	0,18	1,0	2385,66	417,491	1
2	0,600	0,48	1,0	2385,66	1133,189	2
3	0,716	0,66	1,0	2385,66	1569,765	3
4	0,716	0,72	1,0	2385,66	1708,133	4
5	0,716	0,72	1,0	2385,66	1708,133	5
6	0,716	0,72	1,0	2385,66	1708,133	6
7	0,716	0,72	1,0	2385,66	1708,133	7
8	0,716	0,72	1,0	2385,66	1708,133	8
9	0,716	0,72	1,0	2385,66	1708,133	9
10	0,716	0,72	1,0	2385,66	1708,133	10
11	0,600	0,66	1,0	2385,66	1569,765	11
12	0,350	0,48	1,0	2385,66	1133,189	12
13	0,000	0,18	1,0	2385,66	417,491	13

Table 1.3: Spring Constant

Chapter4: Calculation**11) SUMMARY OF FORCES****A) Dead Load****1) on top of slab**

a) Superimposed Load	=	0,000 t/m ²
----------------------	---	------------------------

b) Dead Load of Slab	+	haunch	=	1,743 t/m ²
----------------------	---	--------	---	------------------------

B) on bottom of slab

a) Dead Load of Slab	+	haunch	=	1,868 t/m ²
----------------------	---	--------	---	------------------------

b) Dead Load of Wall	=	1,681 t/m ²
----------------------	---	------------------------

D) Live Load for Rail	=	1,800 t/m ²
------------------------------	---	------------------------

E) Earth Pressure at Active (WALL)

a) Earth Pressure at c/c of top slab	=	0,191 t/m ²
--------------------------------------	---	------------------------

b) Earth Pressure at c/c of bottom slab	=	2,94 t/m ²
---	---	-----------------------

F) Earth Pressure at Rest (WALL)

a) Earth Pressure at c/c of top slab	=	0,309 t/m ²
--------------------------------------	---	------------------------

b) Earth Pressure at c/c of bottom slab	=	4,75 t/m ²
---	---	-----------------------

G) Live Load Surcharge	=	3,326 t
-------------------------------	---	---------

H) Longitudinal Forces

a) Longitudinal Loads	=	2,370 t
-----------------------	---	---------

I) Upward pressure for Dead Load

Wt of. Embankment	=	0	KN/m ²
-------------------	---	---	-------------------

Wt of ballast, rail, sleeper	=	18,141	KN/m ²
------------------------------	---	--------	-------------------

Wt of top haunch	=	1,16	KN/m ²
------------------	---	------	-------------------

Wt of top slab	=	15,94	KN/m ²
----------------	---	-------	-------------------

Wt of wall	=	16,49	KN/m ²
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Wt of bottom haunch	=	1,16	KN/m ²
---------------------	---	------	-------------------

Wt of bottom slab	=	17,17	KN/m ²
-------------------	---	-------	-------------------

Total Upward Pressure	=	70,0555	KN/m²
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J) Upward pressure for Dead Load + Live Load

Upward pressure for Dead Load	=	70,0555	KN/m ²
-------------------------------	---	---------	-------------------

Live load intensity top slab	=	17,656	KN/m ²
------------------------------	---	--------	-------------------

Total Upward Pressure	=	87,71162	KN/m²
-----------------------	---	-----------------	-------------------------

K) Upward pressure without Bottom Slab	=	70,54	KN/m²
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Chapter4: Calculation

12) Load Vaculation for Safe bearing capacity

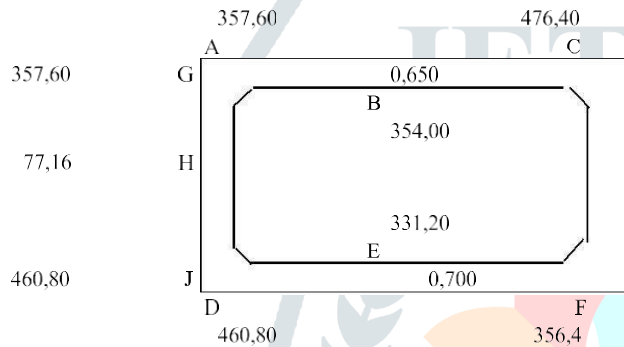
12.1) Total load on Raft

a) Wt of. Embankment	=	0,000	KN/m ²
b) Wt of ballast, rail, sleeper	=	1,849	KN/m ²
c) Wt of top haunch	=	0,12	KN/m ²
d) Wt of top slab	=	1,63	KN/m ²
e) Live load intensity top slab	=	1,800	KN/m ²
f) Wt of wall	=	1,68	KN/m ²
g) Wt of bottom haunch	=	0,12	KN/m ²
f) Therefore, SBC required	=	7	t/m ²

12.2) Design Constant

i) Concrete	=	M 35
ii) Steel	=	Fe 500

12.3) Calculation of Reinforcement



ii) Reinforcement design

Concrete	=	M 35	N/mm ²
Steel	=		N/mm ²
Ultimate B.M.	=	500	
B	=	354,00	KN.m
	=	1000	mm
D	=	650	mm
d eff	=	650 - 75 - 25	
		2	
d eff provided	=	562,5	mm
d eff required	=	280,48	mm
		563 > 280,48	mm
		Hence OK	

iii) Steel requirements

$$M_u = 0.87 \cdot f_y \cdot A_{st} \cdot z \quad \text{Where } z = d \cdot (1 - (1.1 f_y \cdot A_{st} / f_{ck} \cdot b \cdot d))$$

The above is a quadratic eq with A_{st} as variable, $P \cdot S_q (A_{st}) + Q \cdot A_{st} + R = 0$

$$P = (0.957 \cdot S_q(f_y) / (b \cdot f_{ck})) \quad Q = (0.87 \cdot f_y \cdot d) \quad R = M_u$$

$$A_{st} = \{ \pm Q / \sqrt{((S_q Q) - 4 \cdot P R)} \} / (2 \cdot P)$$

	M_u	=	354	b	=	1000
	d	=	562,5	f_y	=	500
Hence,	P	=	6,84	Q	=	-2E+05
	f_{ck}	=	35			

Chapter4: Calculation

BOX DESIGNED FOR ENVELOPE

Points	B.M. KN.m	Factor	B.M. KN.m	S.F. KN	Location	Deff req mm	Deff pro. mm	Dprd mm	Ast req cm ²	Ast prd cm ²	OK/ NOT OK	OK/ NOT OK
Top												
A	298	1,2	357,6	327	Support	377,09	562,5	650	15,27	49,06	OK	OK
B	295	1,2	354	13,2	Mid	375,19	562,5	650	15,10	49,06	OK	OK
C	397	1,2	476,4	354	Support	435,25	562,5	650	20,66	49,06	OK	OK
Bottom												
D	384	1,2	460,8	323	Support	428,06	612,5	700	19,94	49,06	OK	OK
E	276	1,2	331,2	78,3	Mid	362,91	612,5	700	14,09	49,06	OK	OK
F	297	1,2	356,4	343	Support	376,46	612,5	700	15,21	49,06	OK	OK
Wall												
G	298	1,2	357,6	227	Support	377,09	612,5	700	15,27	49,06	OK	OK
H	64,3	1,2	77,16	14	Mid	175,16	615	700	3,18	31,40	OK	OK
J	384	1,2	460,8	358	Support	428,06	612,5	700	19,94	49,06	OK	OK

12.4) Reinforcement Provided

Bar Marked	Dia (mm)	Spacing (mm)	Area cm ²	Remark
a1	25	200	24,53	Vertical wall outer
a2	25	200	24,53	Top bar of top slab
b	25	100	49,06	Bottom bar of top slab
c	25	200	24,53	Bottom bar of bottom slab
d	25	100	49,06	Top bar of Bottom slab
e	20	100	31,40	Inner bar of Wall
f1	25	200	24,53	Top haunch bar
f2	25	200	24,53	Bottom haunch bar
g1	10	200	3,93	Shear link for Slab
g2	10	200	3,93	Shear link for Raft
g3	10	200	3,93	Shear link for Wall
h	12	100	11,30	Distribution Outer wall
h	12	100	11,30	Distribution Bottom slab

Table 1.4: Reinforcement Provided as Per Calculation

Minimum steel required = 0,2 % of Cross-sectional area.
 (IRS Concrete Bridge Code - 1997 Clause no 15.9.4.1) Page no V-72

vertical Main steel Ast min = 0,15 x 100 x 65
 Inside Ast min = 9,75 cm²
 Ast provided = 12 mm 100 mm c/c

= 11,30 cm² in single layer
 Total Distribution = 22,61 > 9,75 Ast min **Hence OK**

12.6) Check for shear

Shear force V = 354,0 KN

Area of steel continuing beyond the point of curtailment
 At = 49,06 mm²

Percent area of steel = $\frac{A_{st}}{b} \times \frac{100}{d}$

Percent area of steel = $\frac{49,0625}{100} \times \frac{100}{650}$
 = 0,08 % 8

$\beta = \frac{0,8\sigma_{ck}}{53,84} / \frac{6,89P_t}{1}$

Where $\tau_c = (0,85/6\beta) * \text{SQRT}(0,80\sigma_{ck}) * (\text{SQRT}(1 + 5\beta) - 1)$
 = 0,21 N/mm²

Shear strength of concrete for 0,08 % tension steel $\tau_c = 0,21$

Nominal shear stress $\tau_v = \frac{354}{1000} \times \frac{1000}{650}$
 = 0,54 N/mm²

Maximum permissible shear stress concrete = 2,3 N/mm²
 (IRC 21 -2000, Page No. 36 Table 12A)
 > 0,54 N/mm² **Hence OK**

Design shear stress $\tau_{us} = 0,54 - 0,21$
 = 0,33 N/mm² **Provide Shear Reinforcement**

2 Legged 10 mm 200 c/c

As prd = 2 x 78,5
 = 1,57 cm²

Minimum area of shear reinforcement

A s > $\frac{0,4}{0,87} \times \frac{1000}{500} \times \frac{100}{200}$

$$= 0,005 \text{ cm}^2 < 1,570 \text{ cm}^2 \quad \text{Hence OK}$$

Spacing of shear stirrups x

$$\begin{aligned}
 &= 0.87 \sigma_y A_{sv} / \tau_{us} * b && [\text{IS 456 -2000,} \\
 &= 207,16 \text{ mm} && \text{Clause No - 40.4,} \\
 &> 100 \text{ mm} && \text{Page No.72]} \\
 &< 0.75 d = 440,625 \text{ mm} \\
 &< 300 \text{ mm}
 \end{aligned}$$



DESIGN OF RETURN WALL

1) Allowable Stresses IRC: 112-2011, A4.5.1, Table A4.2, page no. 253

I) Reinforcement (Normal Condition)

Flexure Tension	=	240 N/mm ²	x	10.2	2446 Kg/cm ²
Shear	=	240 N/mm ²	x	10.2	2446 Kg/cm ²
Direct Compression	=	205 N/mm ²	x	10.2	2090 Kg/cm ² 8;

II) Concrete M35

Permissible direct compressive stress	=	8.33 N/mm ²	x	10.2	85 Kg/cm ²
Permissible flexural compressive stress	=	11.53 N/mm ²	x	10.2	118 Kg/cm ²

III) Tensile stresses

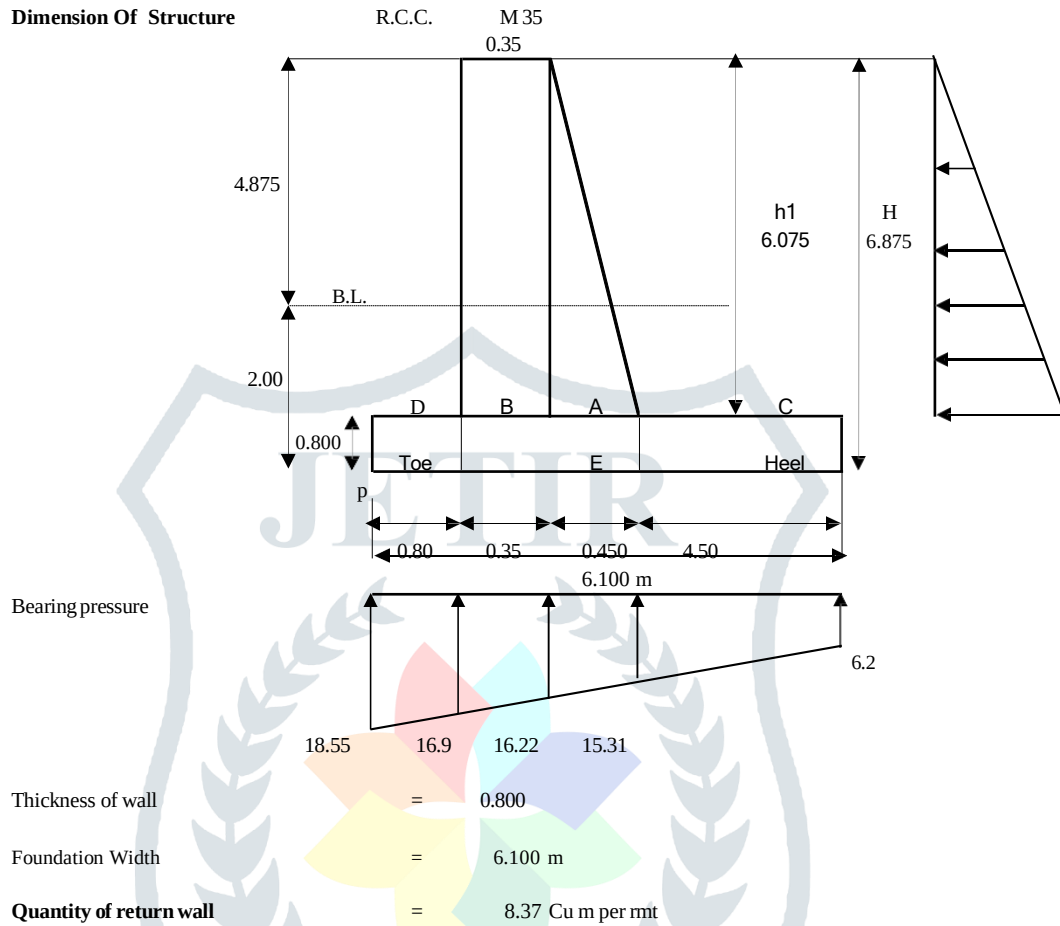
Permissible flexural tensile stress	=	0.67 N/mm ²	x	10.2	6.8 Kg/cm ²
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IV) Modular Ratio

	=	10			
Coefficient of Thermal Expansion	=	1.17 10 ⁻⁵		per degree celsius	

Design for Retaining wall

1) Dimension Of Structure



2) Summary of Safty Factors

Retaining wall is designed considering the formation level at the top of retaining wall.

F.O.S. against overturning	=	4.27	>	2	Hence OK
F.O.S. against Sliding	=	1.60	>	1.5	Hence OK
Base Pressure At Toe	=	18.55 t/m ²			
Base Pressure At Heel	=	6.19 t/m ²			

3) DL of Retaining Wall

SN.	ITEM	Geometr Factor	L m	B m	H m	Vol m ³	Unit Wt. t/m ³	Load (W) 100% t
i)	Section B	1	1.0	0.350	6.075	2.1	2.5	5.32
ii)	Section A	0.5	1.0	0.450	6.075	1.4	2.5	3.42
						3.49		8.7
iii)	Foundation E	1	1.0	6.100	0.800	4.9	2.5	12.20
						8.37		20.93

4) Earth fill on Back

SN.	ITEM	Geometr Factor	L m	B m	H m	Vol m ³	Unit Wt. t/m ³	Load (W) 100% t
i)	Soil on A	0.5	1.0	0.450	6.075	1.4	1.9	2.60
ii)	Soil on C	1	1.0	4.500	6.075	27.3	1.9	51.94
								54.54

5) Center of Gravity at Point P

SN	Item	Self c.g.	Bottom footing
		m	m
i)	Section B	0.175	0.975
ii)	Section A	0.150	1.300
iii)	Foundation E	3.050	3.050
iv)	Soil on A	0.300	1.450
v)	Soil on C	2.250	3.850

6) Stabilising moment

SN	Item	Load W	Lever arm ft. bot.	Bottom Moment
		t	m	t.m.
i)	Section B	5.3	1.0	5.18
ii)	Section A	3.4	1.3	4.44
iii)	Foundation E	12.2	3.1	37.21
iv)	Soil on A	2.6	1.5	3.77
v)	Soil on C	51.94	3.9	199.97
		75.5		250.6

7) Calculation of Active Earth Pressure

Angle of repose	Φ	30 °
Angle of wall friction	δ	20 °
Inclination earth face of wall	α	90 °
Slope of Back fill Surcharge	β	30 °
angle	i	0 °
Dry Density of Backfill	γ	1.9 t/m ³
Density of water	γ_{wat}	1 t/m ³
Saturated Density of Backfill	γ_{sat}	2 t/m ³
coefficient of Friction		0.45

Coefficient of Active earth pressure
From Coulomb's theory of active earth pressure

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \cdot \sin(\alpha - \delta) \cdot \left[1 + \frac{\sin(\phi + \delta) \cdot \sin(\phi - i)}{\sin(\alpha - \delta) \cdot \sin(\alpha + i)} \right]^2}$$

$$K_a = 0.297$$

$$\text{Horizontal co-efficient, } K_{ah} = K_a \cos \delta = 0.279$$

Horizontal component due to friction

$$K_h = 0.279$$

Horizontal component due to inclination of wall

$$\begin{aligned} k_{aH} &= k_a \times \cos 45 \\ &= 0.28 \times 0.707 \\ &= 0.20 \end{aligned}$$

Vertical component

$$\begin{aligned} K_v &= K_a \times \sin \delta \\ &= 0.2 \times 0.34 \\ &= 0.068 \end{aligned}$$

7.1) Calculation of Passive Earth Pressure

Angle of repose	Φ	30 °
Angle of wall friction	δ	20 °
Inclination earth face of wall	α	90 °
Slope of Back fill	β	30 °
Surcharge angle	i	0 °
Dry Density of Backfill Density	γ	1.9 t/m ³
of water	γ_{wat}	1 t/m ³
Saturated Density of Backfill	γ_{sat}	2 t/m ³
coefficient of Friction		0.45

Coefficient of Active earth pressure

From Coulomb's theory of active earth pressure

$$K_p = \frac{\sin^2(\alpha - \phi)}{\sin^2 \alpha \cdot \sin(\alpha + \delta) \left[1 + \frac{\sin(\phi + \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \delta) \cdot \sin(\alpha + \beta)} \right]^2}$$

$$K_p = 0.220$$

$$\text{Horizontal co-efficient, } K_{ph} = K_p \cos \delta = 0.207$$

Horizontal component due to friction

$$K_h = 0.207$$

Horizontal component due to inclination of wall

$$\begin{aligned} k_{aH} &= k_a \times \cos 45 \\ &= 0.21 \times 0.707 \\ &= 0.15 \end{aligned}$$

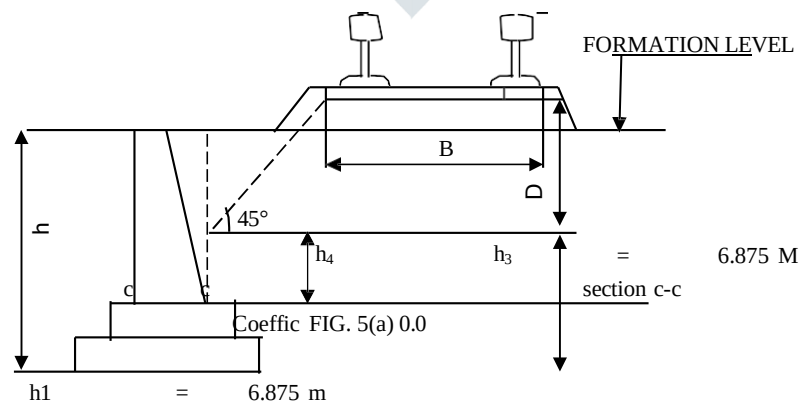
Vertical component

$$\begin{aligned} K_v &= K_a \times \sin \delta \\ &= 0.1 \times 0.34 \\ &= 0.050 \end{aligned}$$

pressure due to live load a

(As per Clause no.5.8.3 Bridge sub - structures & foundation code Page no. IIA - 14)

$$\text{Surcharge ht.} = 6.875 \text{ M from bottom}$$



Surcharge pressure at Founding level

$$\begin{aligned}
 P3 &= \frac{[S + V]}{[B + 2D]} * Ka * h_1 \\
 &= \frac{13.7 + 6.5}{3 + 2} * 0.279 * 6.875 \\
 &= 4.55 \text{ t}
 \end{aligned}$$

This pressure will be assumed to be acting at distance of $h_1/2$.

$$H3 = 1.495 \text{ M}$$

$$\text{act at} = h_3/2$$

$$h = 0.7475 \text{ m}$$

Surcharge pressure at footing top level

$$h_4 = 1.495 - 0.000 = 1.495 \text{ m}$$

$$\begin{aligned}
 P4 &= \frac{[S + V]}{[B + 2D]} * Ka * h_1 \\
 &= \frac{13.7 + 6.5}{3 + 2} * 0.279 * 1.495 \\
 &= 0.99 \text{ t}
 \end{aligned}$$

This pressure will be assumed to be acting at distance of $h_1/2$. act at =

$$h_4/2$$

$$h = 0.748 \text{ m}$$

8) Earth pressure and force up to top of foundation

SN.	Item		Ka	Density	h	Force
				t/m ³		Ka x γ x h
1)	Footing Top	P1	0.297	1.9	6.075	3.43
2)	Surcharge	P2	0.297	1.9	1.200	0.68

SN.	Item		Factor	h	c.g.	c.g.	Force	Moment
					factor			t.m
1)	Footing Top	P1	0.50	6.075	0.42	2.552	10.424	26.60
2)	Surcharge	P2	1.00	6.075	0.5	3.038	4.1181	12.51
								39.11

8.1) Earth pressure and force at bottom of foundation

SN.	Item		Ka	Density	h	Force
				t/m ³		Ka x γ x h
1)	Footing Bottom	P1	0.297	1.9	6.875	3.88
2)	Surcharge	P2	0.297	1.9	1.200	0.68
3)	Surcharge	P3				4.55
4)	Surcharge	P4				0.99

SN.	Item		Factor	h	c.g.	c.g.	Force	acts at 0.42*h	Moment
					factor				t.m
1)	Footing Bottom	P1	0.50	6.875	0.42	2.888	13.350	2.888	38.55
2)	Surcharge	P2	1.00	6.875	0.5	3.438	4.660	0.600	16.02
3)	Surcharge	P4					4.552	0.748	3.40246
4)	Surcharge	P4					0.990	0.748	0.740
							23.552		58.71

9) F.O.S. against Overturning

$$= \frac{\text{Stabilizing moment}}{\text{Overturning moment}}$$

$$= \frac{\sum M_R}{\sum M_O} = \frac{250.57}{58.71}$$

$$= 4.27 > 2 \quad \text{Hence OK}$$

10) F.O.S. against Sliding

$$= \frac{\mu R_v}{R_H} = \frac{75.47 * 0.5}{23.552}$$

$$= 1.602 > 1.5 \quad \text{Hence OK}$$

12) Calculation of eccentricity at foundation level

$$\text{c.g. of resultant (x)} = \frac{\sum M}{\sum V} = \frac{250.6}{75.47} = 3.32 \text{ m}$$

$$\text{eccentricity (e)} = \frac{b}{2} - x$$

$$= \frac{6.10}{2} - 3.32$$

$$= 3.05 - 3.32$$

$$= -0.27 \text{ m}$$

$$\frac{b}{6} = \frac{6.10}{6} = 1.02 \text{ m}$$

$$e < \frac{b}{6} \quad \text{there is no tension.}$$

$$0.508 < 1.02 \text{ m} \quad \text{Hence OK}$$

No area of foundation is in tension.

13) The pressure at the base due to moment

$$\begin{aligned} \text{i) } P_{\max} &= \frac{\sum V}{b} \left[1 + \frac{6e}{b} \right] \\ &= \frac{75.47}{6.10} \left[1 + \frac{6 * 0.51}{6.100} \right] \\ &= 18.552 \text{ t/m}^2 \end{aligned}$$

$$\begin{aligned} \text{ii) } P_{\min} &= \frac{\sum V}{b} \left[1 - \frac{6e}{b} \right] \\ &= \frac{75.47}{6.100} \left[1 - \frac{6 * 0.51}{6.10} \right] \\ &= 6.19 \text{ t/m}^2 \end{aligned}$$

$$\text{iii) } P/A = \frac{75.47}{6.10}$$

$$= 12.37 \text{ t/m}^2$$

$$\text{iv) } (P/a) + (M/Z) = 18.55 \text{ t/m}^2$$

$$(P/a) - (M/Z) = 6.19 \text{ t/m}^2$$

$$\text{S.B.C. required} = 18.55 \text{ t/m}^2$$

14) Design of Return wall

a) Concrete Mix	=	M 35
Steel	=	Fe-500 grade HYSD bars
F _{ck}	=	35 N/mm ²
B	=	1000 mm
D	=	800 mm
Dia	=	20 mm
Cc	=	75 mm

b) Permissible Stresses

F _{cbc}	=	11.53	N/mm ²
F _{st}	=	200.0	N/mm ²
m	=	10	
n	=	$\frac{1}{1 + \frac{F_{st}}{m * F_{cbc}}}$	
n	=	0.37	
j	=	$1 - \frac{n}{3}$	
j	=	0.88	
Q	=	0.5	x n x j * F _{cbc}
Q	=	1.85	N/mm ²

c) Steel

Total B.M.	=	39.11	t.m.
B.M. / per m width	=	39.11	t.m.
Q	=	18.87	kg/cm ²
B	=	100	cm
D	=	80	cm
d _{eff}	=	80	cm
d _{eff} Prov.	=	71.5	cm
d _{eff} Req.	=	45.52	cm
A _{st} req	=	31.14	cm ²
Dia	=	2	cm
A	=	3.14	cm ²

Minimum steel required is 0.2 % of Cross sectional area.

(As per IRC 78 - 2000 Page No. 25 Cl. No. 707.2.7)

Main steel

A _{st} min	=	0.2	x	100	x	80
A _{st} min	=	16	cm ²			
No.	=	5.10				
sp	=	196.25	mm			

d) Vertical Main steel (earth face)

Provide	=	25	mm	100	mm	c/c
A _{st} provided	=	49.06	cm ²	>	16	A _{st} min
				>	31.14	A _{st} req

Hence OK

0.3 Main steel required (other face) = 9.34 cm²

Provide	12	mm @	100	mm c/c	=	11.30	cm ²
					>	9.3	A _{st} req

Hence OK

$$\begin{aligned}\text{Distribution Reinforcement} &= 0.13 * \frac{80 * 100}{100} \\ &= 10.4 \text{ cm}^2\end{aligned}$$

$$\begin{aligned}\text{Distribution Reinforcement (earth face)} &= \frac{3}{3} * A_{st} \\ &= \frac{3}{3} * 10.40 \\ &= 10.4 \text{ cm}^2\end{aligned}$$

$$\begin{aligned}\text{Provide } 12 \text{ mm @ } 100 \text{ mm c/c} &= 11.30 \text{ cm}^2 \\ &> 10.4 \text{ Hence OK}\end{aligned}$$

$$\begin{aligned}\text{Distribution Reinforcement (other face)} &= \frac{1}{3} * A_{st} \\ &= \frac{1}{3} * 10 \\ &= 3.47 \text{ cm}^2\end{aligned}$$

$$\begin{aligned}\text{Provide } 10 \text{ T @ } 100 \text{ mm c/c} &= 7.85 \text{ cm}^2 \\ &> 3.47 \text{ cm}^2 \text{ Hence OK}\end{aligned}$$

$$\begin{aligned}\text{Total Horizontal force} &= 23.55 \text{ t} \\ \text{per m width} &= 23.55 \text{ t} \\ \text{Shear stress} &= \frac{23552}{80 * 100} \\ &= 2.94 \text{ Kg/cm}^2\end{aligned}$$

Max Shear stress of Concrete due to Reinforcement
(As per IRC 21 - 2000 Page No. 36 Cl. No. 304.7.1.2 , table 12A)

$$\begin{aligned}&= 22 \text{ Kg/cm}^2 \\ \text{Max Shear stress of Concrete due to Reinforcement} & \\ \begin{array}{ccc} \text{pt} & \text{tc} & \% \\ 0.25 & 0.23 & \\ 0.613 & 0.33 & \\ 0.75 & 0.37 & \end{array} & \\ T_c &= 3.317 \text{ Kg/cm}^2 > 2.94 \text{ Kg/cm}^2\end{aligned}$$

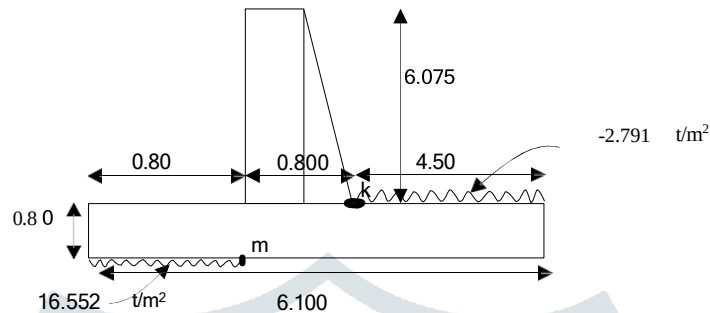
Nominal Stirrups are required

$$\begin{aligned}&5.00 \text{ Legged } 8 \text{ mm } 200 \text{ C/C} \\ \text{As prov.} &= \frac{1000}{200} * 5.00 * 50.24 \\ &= 12.56 \text{ cm}^2\end{aligned}$$

Minimum area of shear reinforcement

$$\begin{aligned}A_s &> \frac{0.4}{0.87} * \frac{1000}{500} * 200 \\ &= 1.839 \text{ cm}^2 < 12.560 \text{ cm}^2 \text{ Hence OK}\end{aligned}$$

vi) Design for Foundation

**B.M at k**

Wt of Soil	6.075×1.90	=	11.54	t/m ²
Wt of Footing	0.80×2.50	=	2.00	t/m ²
Total		=	13.54	t/m ²

$$\text{Upward Pressure} = 10.75 - 13.54 = -2.79 \text{ t/m}^2$$

$$\text{B.M at k} = \frac{wl^2}{2} = \frac{-2.79 \times 4.50^2}{2} = -28.26 \text{ t.m}$$

B.M at m

$$\text{Upward Pressure} = 18.55 - 2.00 = 16.552 \text{ t/m}^2$$

$$\text{B.M at m} = \frac{wl^2}{2} = \frac{16.55 \times 0.80^2}{2} = 5.297 \text{ t.m}$$

c) Steel At Top

Total B.M.	=	28.26	t.m.
B.M. / per m width	=	28.26	t.m.
Q	=	18.87	kg/cm ²
B	=	100	cm
D	=	80	cm
d eff	=	80	- 7.5 - 2
d _{eff} Prov.	=	71.5	cm 2
d _{eff} Req.	=	38.70	cm < 71.5 cm Hence OK
Ast req	=	22.50	cm ²
Dia	=	2	cm
A	=	3.14	cm ²

Minimum steel required is 0.2 % of Cross-sectional area.

(As per IRC 78 - 2000 Page No. 25 Cl. No. 707.2.7)

Main steel

Ast min	=	0.2	x	100	x	80
Ast min	=	16	cm ²			
No.	=	5.10				
sp	=	196.25	mm			

d) Main steel At Top

Provide	=	20	mm	100	mm	c/c	
	=	0	mm	100	mm	c/c	Extra Bar at Top

Ast provided	=	31.40	cm ²	>	16.00	Ast min	
				>	22.50	Ast req	Hence OK

Distribution Reinforcement At Top & Bottom	=	0.12	*	$\frac{80 * 100}{100}$			
	=	9.6	cm ²				

Provide	12	mm @	100	mm c/c	=	11.30	cm ²
Total Distribution					=		
						22.608	cm ²
					>	9.6	cm ² Hence OK

c) Steel At Bottom

Total B.M.	=	5.30	t.m.				
B.M. / per m width	=	5.30	t.m.				
Q	=	18.87	kg/cm ²				
B	=	100	cm				
D	=	80	cm				
d eff	=	80		-	7.5	-	1.6
d _{eff} Prov.	=	71.7	cm				2
d _{eff} Req.	=	16.75	cm	<		71.7	cm Hence OK

Ast req	=	4.22	cm ²				
Dia	=	1.6	cm				
A	=	2.01	cm ²				
Minimum steel required is		0.2	% of Cross-sectional area.				
(As per IRC 78 - 2000 Page No. 25 Cl. No. 707.2.7)							
Main steel							
Ast min	=	0.2	x 100	x	80		
Ast min	=	16	cm ²				
No.	=	7.96					
sp	=	125.60	mm				

d) Main steel At Bottom

Provide	=	16	mm	100	mm	c/c	
Ast provided	=	20.10	cm ²	>	16.00	Ast min	
				>	4.22	Ast req	Hence OK
Total Horizontal force	=	13.54	t				
per m width	=	13.54	t				
Shear stress	=	$\frac{13543}{80.000 \times 100}$					
	=	1.69	Kg/cm ²				

Max Shear stress of Concrete due to Reinforcement							
(As per IRC 21 - 2000 Page No. 36 Cl. No. 304.7.1.2 , table 12A)							
	=	22	Kg/cm ²				

Max Shear stress of Concrete due to Reinforcement							
pt	=	1.712	%				
Tc	=	2.3	Kg/cm ²	>	1.69	Kg/cm ²	

Nominal Stirrups are required

$$\begin{aligned}
 &10.00 \quad \text{Legged} \quad 8 \quad \text{mm} \quad 200 \quad \text{C/C} \\
 \text{As prov.} &= \frac{1000}{200} \times 10.00 \times 50.24 \\
 &= 25.12 \text{ cm}^2
 \end{aligned}$$

Minimum area of shear reinforcement

$$\begin{aligned}
 A_s &> \frac{0.4}{0.87} \times \frac{1000}{500} \times 200 \\
 &= 1.839 \text{ cm}^2 < 25.120 \text{ cm}^2 \quad \text{OK}
 \end{aligned}$$

x) **Summary of Steel**

i) **At wall**

Earth filled soil side

Verticals	25 mm @	100 mm	c/c
Horizontal	12 mm @	100 mm	c/c

Bar marked in drg

(7)
(8)

Outside

Verticals	12 mm @	100 mm	c/c
Horizontal	10 mm @	100 mm	c/c

(9)
(10)

Links along Height 8 mm @ legged Stirrups 200 mm

(11a)

Links along Length 8 mm @ legged Stirrups 200 mm

(11)

ii) **At Foundation**

At Bottom

16 mm @	100 mm	c/c
12 mm @	100 mm	c/c

(1)
(2)

At Top

20 mm @	100 mm	c/c
12 mm @	100 mm	c/c

(3)
(4)

Links along Width 8 mm 200 mm c/c

(5a)

Links along Length 8 mm 200 mm c/c

(5)

17) Crack Width Calculation

Height	=	1.00 m
Length	=	1.0 m
The thickness	=	0.8 m

i) Ultimate limit state: Flexure

Provided clear cover to any reinforcement	=	0.05 m
Grade of concrete used	=	35
Characteristic yield strength of reinforcement	=	500 MPa
Mean value of axial tensile strength of concrete	f_{ctm} =	2.8 MPa
Design value of concrete compression strength	f_{cd} =	15.86 MPa
Effective depth of provided section	=	0.715 m
Minimum breadth of section over the depth	b_w =	1000 mm
Effective depth at critical section or face of bearing	d =	800 mm
Longitudinal Tensile reinforcement provided	A_{st} =	4906 mm ²
Cross sectional area of concrete	A_c =	800000 mm ²
Applied longitudinal Force	N_{ed} =	0 N
Shear reduction factor for concrete cracked in shear	v_1 =	0.532
Inclination of failure plane in radians	θ =	0.79
Lever arm (can be taken as 0.9d for RCC section)	z =	720 mm

iii) Serviceability limit state: Stress check

Width of the section at the centroid of the tension steel	b_t =	1000 mm
Effective depth of provided section	d =	715 mm
Permissible Compressive Stress for Concrete in bending	σ_{cbc} =	16.8 MPa
Permissible Compressive Stress for Steel	σ_{sc} =	400 MPa
Permissible Tensile Stress for Steel	σ_{st} =	350 MPa
Modulus of elasticity of steel	E_s =	200000 MPa
Modulus of elasticity of concrete	E_c =	34000 MPa
Modular ratio E_s/E_c or $280/3\sigma_{cbc}$	m =	8.00
Area of tension reinforcement	A_s =	4906 mm ²

Depth of neutral axis from outer compression fibre can be calculated by equating the area moment of concrete in compression and equivalent steel in tension about neutral axis. Rearranging this gives a quadratic equation in d_c which can be solved by finding roots

$$\boxed{\text{Value of } 2mA_s/b_t = 78.5} \Rightarrow \boxed{d_c^2 + 2mA_s/b_t \cdot d_c - 2mA_s/b_t \cdot d = 0}$$

$$\Rightarrow d_c^2 + 78.5 \cdot d_c - 78.5 \cdot d = 0$$

Depth of neutral axis or concrete in compression, $d_c = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$

where, $a = 1$, $b = 78.5$ and $c = -56128$

$d_c = 201 \text{ mm}$ or -279 mm

Knowing the depth of neutral axis, stress in concrete and steel can be calculated using the moment equation. Substituting the design moment & other parameters gives stress in concrete

$$\boxed{M = b_t \cdot d_c \cdot s_c / 2 \cdot (d - d_c / 3)}$$

Total design bending moment	=	39.11 Tm	
	$\Rightarrow$	391053585	= 65092553 s_c
Stress in concrete s_c	=	6.01 MPa	Hence safe

Stresses are assumed to be within linear elastic limit, hence stress in respective concrete fibre and in steel are proportional to their distance from neutral axis

$$\frac{s_c/d_c = s_{st}/m(d - d_c)}{\text{Stress in steel } s_{st} = \quad = \quad 123.0 \text{ MPa} \quad \text{Hence safe}}$$

iv) Serviceability limit state: Crack width (As per Cl: 12.3.4 of IRC 112: 2011)

Live Load surcharge has been removed for calculation of crack width

Negative moment	Max.stress $\sigma_t = - b_1 b \frac{q}{t} =$	39.1 T/m ²
Sagging moment	Max.stress $\sigma_b = b_2 b \frac{q}{t} =$	39.1 T/m
Maximum bending moment at the crack	$M_g =$	39.1 Tm
Cover to longitudinal reinforcement	$C =$	50 mm
C/C spacing between reinforcement	$s =$	100 mm
Area of Longitudinal reinforcement provided	$A_{sl} =$	4906 mm ²
Equiv Dia of steel provided	$\phi_{eff} =$	20 mm
Cracked moment of Inertia	$I_{cr} =$	1.31E+10 mm ⁴
Stress in reinforcement for cracked section	$\sigma_{st} =$	123.0 MPa
2.5 times effective cover to longitudinal reinforcement	$2.5*(h-d) =$	150.0 mm
1/3rd depth of section below neutral axis	$(h-x)/3 =$	199.7 mm
half the depth of total section	$h/2 =$	400.0 mm
Eff area of concrete in tension surrounding reinforcement	$A_{c, eff} =$	150000 mm ²
	$\rho_{p, eff} =$	0.0327
	$e_{sm} - e_{cm} =$	0.00037
	$5*(c+\phi/2) =$	300 mm
Maximum crack spacing	$S_{r, max} =$	274 mm
Crack Width	$W_k =$	0.101 mm
	0.101 < 0.3, Hence OK	

CHAPTER 5

RESULTS AND DISCUSSION

5.1 Results & Discussion (Including Skew Angle Considerations)

5.1.1 Modeling and Analysis Results

The RCC box was modeled in STAAD.Pro with a skew angle of 30° , representing the angle between the centerline of the road and the railway alignment. Skewed configurations are common in railway projects to accommodate alignment constraints.

- Geometry: Clear opening of $6.0 \text{ m} \times 4.5 \text{ m}$, with skew angle of 30° , effective width and length adjusted to account for oblique loading.
- Loading Conditions: Considered Dead Load, Live Load (IRC Class A and 70R), Earth Pressure, Surcharge, Impact Load (as per IRS: CBC), and Water Pressure.
- Support and Soil Conditions: Skew angle required modeling of soil-structure interaction more critically near acute corners. Fixed base assumed, with $\text{SBC} = 250 \text{ kN/m}^2$.

5.2 Analysis Outcomes for Skewed Configuration

5.2.1 Bending Moment and Shear Force:

- Due to the skew, torsional moments and diagonal shear developed at slab-wall junctions.
- Maximum BM in top slab (skewed mid-span): $+230 \text{ kNm}$
- Maximum shear near obtuse corners: 105 Kn

5.2.2 Torsion Effects:

- Torsional moment due to unsymmetrical load path increased by $\sim 12\%$ compared to right-angle configuration.
- Additional diagonal reinforcement required at acute corners.

5.2.3. Deflection:

- Max deflection: 6.8 mm (within permissible limit) but increased compared to non-skewed design.

5.3 Design of RCC Elements with Skew Angle

5.3.1.1 Top and Bottom Slabs:

- Reinforcement designed considering diagonal moments and torsion.
- Top slab: $16 \text{ mm} @ 125 \text{ mm c/c}$ in skewed direction.

5.3.1.2 Side Walls:

- Wall sections near acute angles experienced stress concentration.
- Horizontal bars increased to $20 \text{ mm} @ 150 \text{ mm c/c}$.

5.3.1 Haunch and Corner Detailing:

- Acute corners reinforced with diagonal bars to handle torsional shear.
- 45° haunches provided, thickened at acute corners.

5.4.1 Serviceability and Crack Width Check

- Crack width under critical load combo: 0.22 mm (within limit but higher than right-angled case).
- Extra cover (60 mm) provided at corners and skewed faces for enhanced durability.

5.5.1 Stability Checks (Considering Skew Effects)

- Sliding: FOS = 2.05 (>1.5) → Safe
- Overturning: FOS = 2.15 (>2.0) → Safe
- Bearing Pressure: Max = 192 kN/m² < SBC (250 kN/m²) → Safe
- Extra checks done for eccentric loading due to skew, especially for differential bearing pressure along the length.

5.6.1 Discussion

The introduction of a 30° skew angle significantly influenced the load paths, torsional effects, and reinforcement detailing. Unlike rectangular configurations, skewed RCC boxes demand careful analysis of:

- Load eccentricity and distribution
- Stress concentration at corners
- Effective slab width and shear lag

Software modeling (STAAD.Pro) was essential to capture these behaviors accurately. The structure was verified to be safe and stable, with serviceability within code limits.

5.6.1.1 Key Observations:

- Acute corners are the most critical zones in skewed boxes.
- Torsional and diagonal reinforcement is essential.
- Skew angle increases complexity in detailing and execution.
- It is advisable to use 3D finite element modeling for skew angles $>25^\circ$.

CHAPTER 6

CONCLUSION & FUTURE SCOPE

6.1 Conclusion

The present study focused on the design and structural analysis of a skewed RCC box culvert for a Railway Under Bridge (RUB) on a proposed new railway line. The inclusion of a 30° skew angle presented significant structural challenges compared to conventional right-angled box designs.

Key conclusions from the project are:

- **Skew Angle Impact:** The presence of a skew angle introduced asymmetry in load paths, resulting in increased torsional moments, diagonal shear, and stress concentration, particularly at acute corners.
- **Structural Performance:** Despite the complexity, the RCC box was found to be structurally safe and stable under all loading conditions (DL, LL, earth pressure, surcharge, and impact loads) as per relevant IRC, IRS, and IS codes.
- **Reinforcement Optimization:** Reinforcement had to be enhanced and carefully detailed at skewed junctions, especially to handle torsion and diagonal cracking. Proper detailing of haunches and corner bars ensured serviceability limits were met.
- **Software Utility:** Advanced modeling using STAAD.Pro allowed precise evaluation of skew effects, facilitating effective and economical design while maintaining safety standards.

Chapter 6: Conclusion & Future scope

7.1 Future Scope

The study opens up several possibilities for further exploration and improvement:

- High speed train load modeling , moving load analysis and fatigue life prediction of RCC. Fiber Reinforced concrete , Self healing concrete (crack automatically repair), Geofoam (Reduce earth pressure) and Ultra High Performance concrete .
- Optimization of slab thickness , reinforcement percentage , 3d analysis of skew box culvert , combined bending and torsion study and parameter studies (angle vs stress).
- Rapid construction Techniques Traffic interruption free method , life cycle cost analysis , corrosion resistant steel and derability based design approach
- Install sensors, vibration Sensor , Crack Sensor. Hydraulic capacity redesign , Flood resistant RUB design . drainage optimization.

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