STUDY ON STRUCTURAL ASPECTS OF UNDERPASSES IN SOUTHERN TRANSPORT DEVELOPMENT PROJECT

The thesis submitted to the Department of Civil Engineering of the University of Moratuwa in partial fulfillment of the requirements for the Degree of Master of Engineering in Structural Engineering Design.

By

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ABSTRACT

The Colombo-Matara express highway also known as the Southern Lanka Express Highways or simply the Southern Expressway is a highway currently under construction in Sri Lanka. The 126 km long highway will link the Sri Lankan Capital Colombo with Matara, a major city in the Southern Province of the Island. Construction of the highway began in 2006 and it is expected to be completed in 2010 at the cost of $600 million. When completed, it will reduce the time taken to travel from Colombo to Matara to one and a half hours from the current four hours.

It is known fact that the Southern Highway and other subways linked with are normally supported by a wide range of different structures which require careful thought in selecting a suitable one for each location. In fact, these structures form a vital part of transport infrastructure and the smooth running of the network as designed. Even though, this study has been narrow down only to underpasses from the vide range of structures being used. Therefore, in this research work, it is mainly focused on to the underpasses such as metal and concrete underpasses used in and its significant impact on the cost initiatives, suitability and the environmental impacts and etc.

The technology used for the metal underpasses on this project is new to Sri Lanka. Traditionally in Sri Lanka, pre cast concrete structures are the preferred option, however, in this project, metal underpasses has also been used. The introduction of new technology requires knowledge of their structural behavior, particularly when used in combination with other materials, and their long-term durability. Over the last years, many structures have started to show signs of degradation and deterioration as a result of the high chlorate content in the air in southern Sri Lanka and some kind of crack failures due to bad workmanship as well as lack of adequate supervision. All these issues has been discussed and concluded in this report in a precise manner based on physical observation and on literature survey.

Finally, this research concludes that the use of concrete box underpasses in the southern highway is mostly substantiated with country like Sri Lanka due to its inherent characteristics and with the economy and the durability concerns.

In fact, this report is a part of a post contract analysis which describes important facts that had to be emphasized in selection of the structure underpasses for Southern highway project and concluded which type of underpasses would have been used with the great economic impact to Sri Lanka.

Acknowledgement

I would like to make this opportunity to forward my sincere thanks to the project supervisor, Prof. M.T.R. Jayasingha who helped me to make this project a success by giving advice and looking in to the problems encountered. His guidance and constructive criticism helped me o execute the project successfully.

I wish to thank the Vice Chancellor, Dean of the Faculty of Engineering and the Head, Department of Civil Engineering for allowing me to use the facilities available at the University of Moratuwa.

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I wish to thank to Dr. I.R.A.W. Weerasekara, course coordinator and Dr(Mrs) M.T.P. Hettiarachchi, the research coordinator of the project for the encouragement given to me in completing this study, and all the lecturers of the postgraduate course on Structural Engineering Design who helped me to enhanced my knowledge.

Special thanks go to Mr. L.G. Sirisena, Mrs. V.B. Panditha, my loving parents, for the support given for my education and learn to be confident throughout the career. I also would like to thank my husband for giving valuable support and encouragement to complete the study during the period.

Finally, I gratefully acknowledge everybody who helped me in numerous way in completing my research study.

V.G. Liyanagamage.

August 2009.

DECLARATION

I, V.G. Liyanagamage, hereby declare that the content of the thesis is the output of the original research work carried out at the Department of Civil Engineering, University of Moratuwa. Whenever others' work is included in this thesis, it is appropriately acknowledged as a reference.

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Date : 06/10/2009

UOM Verified Signature

Signature

Name of the Supervisor : Prof. M. T. R. Jayasingha

Date : 06/10/2009

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Chapter 1

1.0 Introduction

1.1 General

Traffic safety concerns, the use of underpasses and Overpasses are now increasingly being adopting in the highway construction in Sri Lanka. When the Southern Transport Development project is concern, the design professionals have adopted these features with a great attention to cater the smooth transport with a safe and without undue delays. In this scenario, various types of underpasses have been integrated along with the road lines and crossings. Those are bridges, metal structures, box culverts and pipe culverts. In the designer's point of view, there are several issues that have to be concerned in selecting a typical underpass to the highway project. Some fundamental issues governing under these criteria are type of road users, cost, method of construction, Procurement of construction materials as well as availability, durability and the environmental issues.

Metal structures and box culverts occupied in roads play a major role as replacement for the bridges as well as the cost significant alternative. Following advantages could have been encountered when such a structure is incorporated with the roads.

- No bridge deck deterioration problems.
- Eliminates constant maintenance of bridge approaches and painting of superstructure.
- Permits use of constant roadway section in the vicinity of structure.
- Roadway easily widened by simple extension of ends.
- Readily available – components are standard shop items – can be field assembled with unskilled labour.
- Less design and construction time – total project completed earlier.
- Environmentally acceptable – permits natural appearance of earth slope and vegetation to be utilized.

In Southern Transport Development Project we mainly identify two types of underpasses. Those are metal structures and box culverts. Fig. 1a shows the front view of a metal structure and Fig. 1b shows view of a box culvert.

Figure 1.1 : View of a Metal Structure

Figure 1.2 : View of a Box Culvert

Metal Underpasses

In relation with the Southern Transport Development project, there are five types of metal underpasses. Curved corrugated metal plates have mostly used for this metal underpasses. Those are MAUP 47N, MAUP 55N, MAUP 67N, HPA 60N, HPA 74N & HES 87N. Figure 1.3 shows the typical drawings of metal structures mentioned above. The selection of the type depends specially on the maximum span & rise. Moreover, the Span & the rise depend on the design elevations of the secondary road which passes through the structure and the type of vehicles using that secondary road.

Structural design of metal underpasses is in accordance with the Standard Practice for Structural Design of Corrugated Steel Pipe , Pipe Arches and Arches for Storm and Sanitary Sewers and Other Buried Applications (ASTM A 796/A 796-01). Dimensions and tolerances is in accordance with Standard Specification for Corrugated Steel Structural Plate, Zinc – coated, for Field – Bolted Pipe, Pipe – Arches and Arches (ASTM A761/A 761M-02).

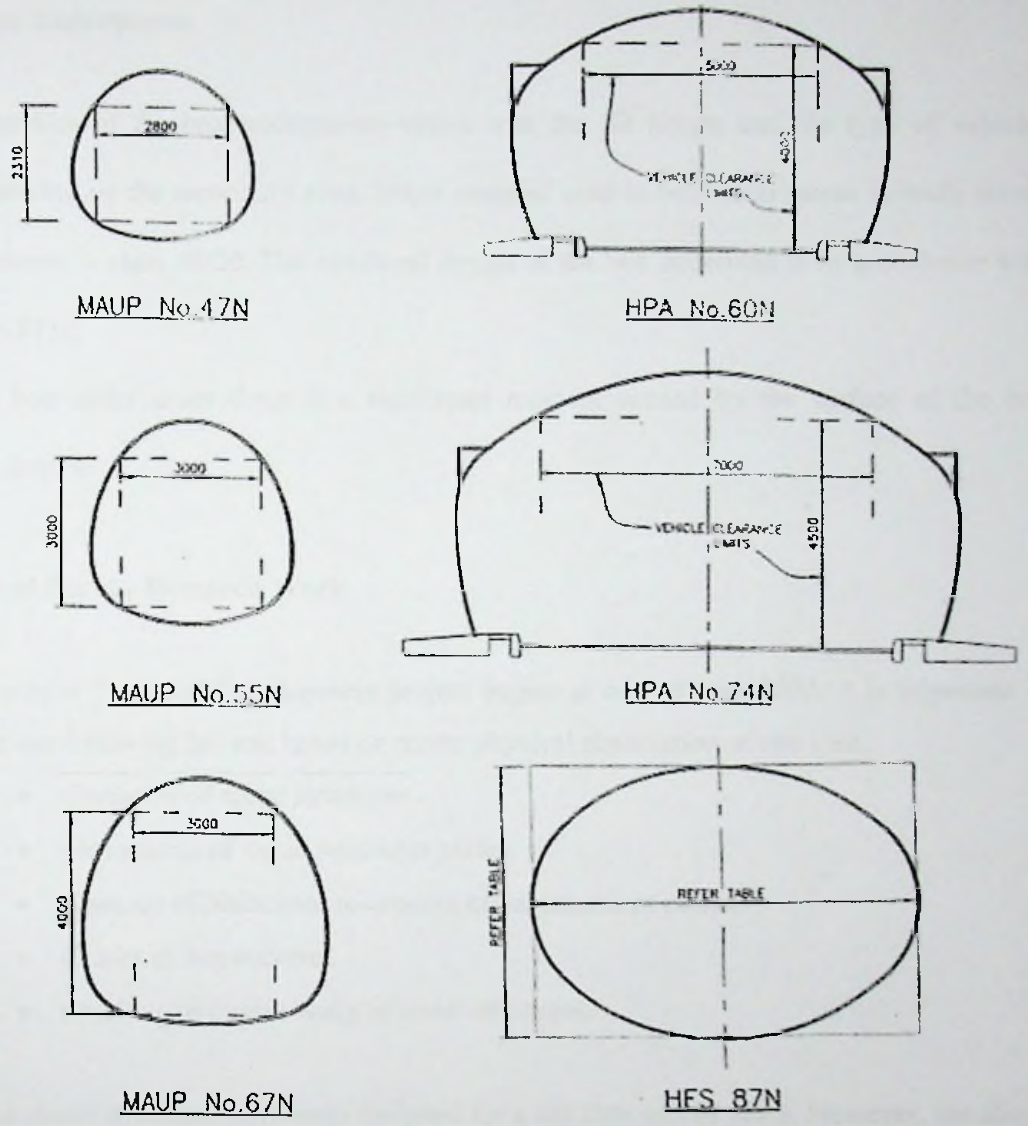


Figure 1.3 : Typical Details of Metal Structures

Box Underpasses

The size of the box underpasses varies with the fill height and the type of vehicles traveling on the secondary road. Major material used in box underpasses is ready mixed concrete – class 30/20. The structural design of the box underpass is in accordance with BS 8110.

In box underpasses there is a significant moment carried by the surface of the box underpass.

Need For the Research Work

Southern Transport Development project begins at mid of year 2002. It is important to list out following failures based on recent physical observation of site visit,

- Corrosion of metal structures .
- Movements of metal structures plates.
- Absence of Nails used to connect metal structures plates.
- Cracks in box culverts.
- Cracking in husker walls of metal structures.

The above structures have been designed for a life time of 100 years. However, the above mentioned defects have been noted with in a period of four years after the construction. Road was not opened to traffic during this period (i.e. structures have not subjected to design loads)

1.2 Main Objectives

To obtain field surveys and analytical research data that can substantiate specification for the selection of most effective type of underpasses in modern highways as a means to reduce wastage occurred due to use of ineffective type of underpasses by considering the structural aspects.

1.3 Methodology

In order to achieve the above objectives, the following methodology was adopted:

1. A literature review was conducted to determine the desirable features that should be adopted in construction of metal underpasses and box culverts.

2. Field survey to evaluate current condition of structures.

3. A comparison was made for the structural forms recommended for the underpasses by means of reaching compromise solutions.

4. A comparison was made by designing the metal structure manually and box culvert in SAP 2000 to find the most effective type of underpass that can be resist vertical and horizontal loads.

5. Cost comparison was carried out between metal structure and box culvert

1.4 Main Findings

The main findings of this research can be presented with respect to structural forms, structural detailing and cost implications.

1 The Construction of Box Culvert is more cheaper than the construction of Metal Structure.

2 Box culvert is more durable than the Metal Structure

1.5 Arrangement of the Report

This report is presented in the following manner.

Chapter 2 presents a detailed literature review made to determine the desirable features that should be adopted in construction of metal underpasses and box culverts.

Chapter 3 deals with the field visits made and the detail description about the site condition.

Chapter 4 presents the design of metal structure and the results

Chapter 5 deals with the analysis of box culvert and the design.

Chapter 6 presents the cost comparison between metal structure and box culvert.

Chapter 7 presents the conclusions made from the research.

Chapter 2

2.0 Literature Review

2.1 Applications of Corrugated steel products

Corrugated steel products have been used for over 75 years play a major role in the modern engineering technology for a wide range of important functions. Flexible steel conduits play an important role in the form of culverts, storm sewers, subdrains, spillways, underpasses , conveyor conduits and service tunnels: for highways, railways, airports.

In the late 1960's , developments were made which involved adding longitudinal and circumferential stiffening members to the conventional corrugation structural plate structures that permitted the use of larger sizes and increased permissible live and dead loads. The above two types of stiffeners are shown in Figure 2.1.

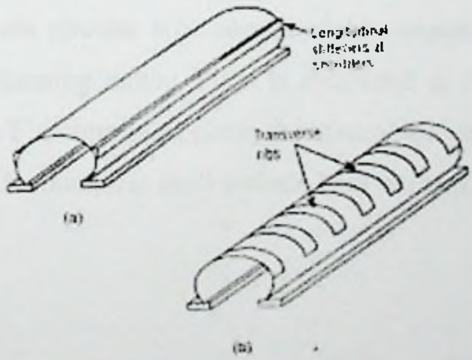


Figure 2.1 : (a) Longitudinal Stiffeners (b) Transverse Stiffeners (Ref. : Design and Construction of Soil Steel Bridges, George Abdel – Sayed)

Some of the applications in which these structures are serving are bridges, highway and railway overpasses or underpasses, stream enclosures, tunnels, culverts and conveyor conduits. These structures have been popular for bridge replacement, and when used as such provide the following advantages:

1. No bridge deck deterioration problems.
2. Eliminates constant maintenance of bridge approaches and painting of superstructure.
3. Permits use of constant roadway section in the vicinity of structure.
4. Readily available – components are standard shop items – can be field assembled with unskilled labour.
5. Less design and construction time – total project can be completed earlier.
6. Less construction engineering and field inspection.
7. Environmentally acceptable – permits natural appearance of earth slope and vegetation to be utilized.
8. Minimum delay to earth moving or other construction operations.
9. Least affected by weather and temperature.

2.2. Description of Corrugations

Types of corrugations available are shown in Figure 2.2. Corrugations commonly used for pipes are termed circular arcs connected by tangents, and are described by pitch, depth and inside forming radius. Pitch is measured at right angles to the corrugations from crest to crest. For corrugated plate, the thickness shall be measured on the tangent of the corrugations. The thickness shall include both the base metal and the coating.

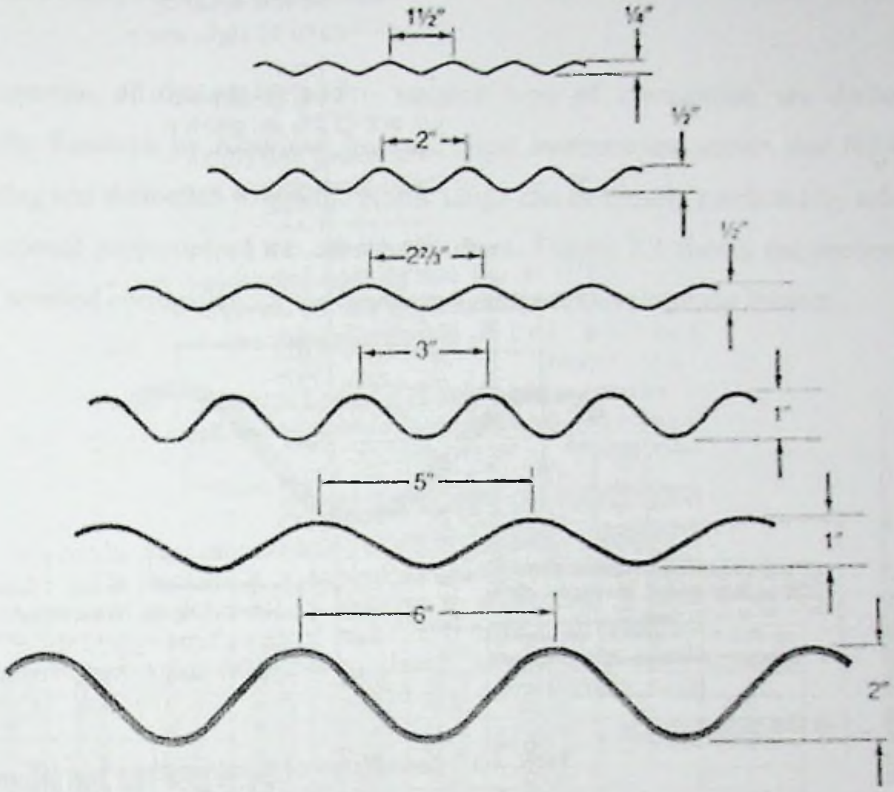
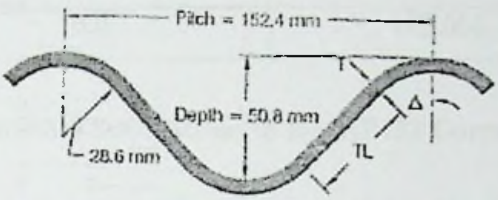


Figure 2.2: Types of Corrugations available

In Southern Transport Development Project they have used 6 by 2 in (152 by 21mm) corrugation. The above corrugation is the Standard of the American Association of State Highway and Transportation officials.

2.3. Structural Properties of Conduit Wall

Sectional properties of the arc – and – tangent type of corrugation are derived mathematically. Research by American Iron and Steel Institute has shown that failure loads in bending and deflection within the elastic range can be closely predicted by using computed sectional properties of the corrugated sheet. Figure 2.3 shows the sectional properties of selected corrugation for the Southern Transport Development Project.



Specified Thickness (mm)	Area of Section,A, mm^2/mm	Tangent Length,T_L, mm	Tangent Angle ,Δ,°	Moment of Inertia,mm^4/mm	Radius of Gyration ,r,mm	Ultimate Strength of Bolted Structural Plate Longitudinal Seams in kN per m of Seam		
						2 Bolts per Corrugations	3 Bolts per Corrugation	4 Bolts per Corrugation
2.82	3.29	48.08	44.47	990.06	17.30	613.00	-	-
3.56	4.24	47.27	44.73	1280.93	17.40	905.00	-	-
4.32	5.18	46.43	45.00	1575.89	17.40	1182.00	-	-
4.79	5.80	45.90	45.18	1769.80	17.50	1357.00	-	-
5.54	6.77	45.03	45.47	2079.80	17.50	1634.00	-	-
6.32	7.74	44.15	47.77	2395.25	17.60	1926.00	-	-
7.11	8.72	43.23	46.09	2717.53	17.70	2101.00	2626.00	2830.00
8.08	9.89	41.99	46.47	3113.54	17.70	-	-	3430.00
9.65	11.88	40.16	47.17	3801.80	17.90	-	-	4159.00

Figure 2.3: Sectional Properties of Selected Corrugation (Ref. : ASTM Designation A 796/A 796 M -01, Table 26)

2.4. Pipe Seams

Standard method of shop – fabricating the seams of annular corrugated steel pipe and pipe – arches are;

- Riveted Seams
- Spot welded seams
- Bolted Seams and joints

In Southern Transport Development Project they used bolted seams of 3/4in diameter (high strength hexagonal bolts meeting ASTM A 449) Following seam strength values are based on a seams using ¾" bolts with heavy hex. heads spaced at 13.12 bolts per meter. These are designed for fitting either the crest or valley of the corrugations, and to give maximum bearing area and tight seams without the use of washers. Table 2.1 describes the ultimate seam strength values for MP 152 corrugated structures.

Specified Thickness (mm)	Seam Strength (kg/m)
2.82	63,217
3.56	84,830
6.0	182,061

Table 2.1 : Ultimate Seam Strength for MP152 Corrugated Structures

2.5 Minimum Cover Requirements

Where pipe is to be places under roads, streets or free ways, the minimum cover requirements shall be determined. Minimum cover is the distance from the top of the pipe to the top of the rigid pavement or to the top of subgrade for flexible pavement.

2.6 Normal Bedding

Pressures developed in the shell by the weight of the backfill and live loads are transmitted both to the side fill and strata underling the pipe. The bedding is the portion of the foundation in contact with the bottom or invert of the structure. Depending upon the size and type of structure, the bedding may either be flat or shaped. With flat bedding, the pipe is placed directly on the fine graded upper portion of the foundation. For pipe – arches and large span structures, with invert plates exceeding 3700mm in radius, the bedding should be shaped to the approximate profile of the bottom portion of the structure. Alternatively, the bedding can be shaped to a slight vee shape. Typical details of flat bedding and vee shaped bedding are shown in Figure 2.4 and Figure 2.5 respectively (Ref. : Installation & Backfilling Standards for Armco Structures).

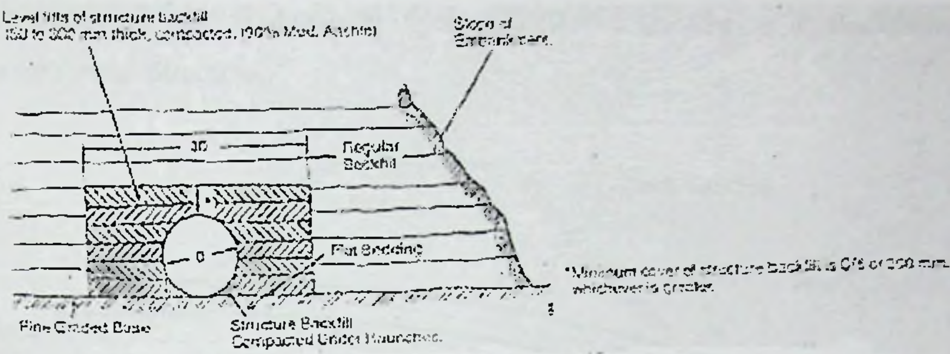


Figure 2.4 : Typical Flat Bedding

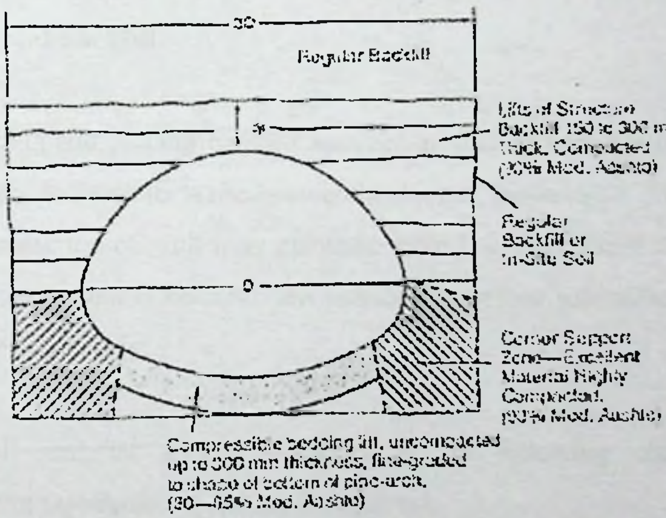


Figure 2.5 : Typical Vee Shaped Bedding

2.7 Camber

The soil cover above the conduit varies along the pipe because of embankment slopes . Due to this uneven cover, the foundation under the pipes settles more under the middle length of the conduit than under the outer length. Longitudinal profile of the bedding must account for this uneven settlement. Camber is simply a rise at the center of a culvert above a straight line connecting its ends to avoid a sag in the longitudinal profile of the culvert. A cambered pipe is shown in figure 2.6 (Ref. : Installation & Backfilling Standards for Armco Structures) .

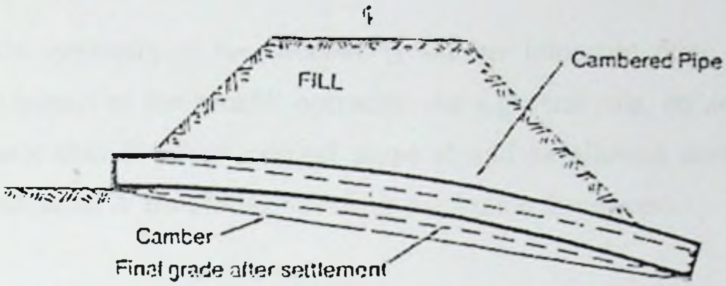


Figure 2.6 : Cambered pipe

2.8 Selection of Structural Backfill

Requirements for selecting and placing backfill material around or near the conduit are similar, in some respects , to those for a roadway embankment. However, a difference in requirements arises because the conduit may generate more lateral pressure than would the earth within the embankment if no structure existed. Therefore soil adjacent to the conduit must be compacted densely.

The Structural backfill material should conform to the following classification (Installation & Backfilling Standards for Armco Structures);

- Minimum Grading Modulus : (G.M.) 0,8
- Maximum Plasticity Index (P.I) 10 + 3 G.M.
- Minimum CBR at compacted density 15%
- Minimum Compacted Density (MOD. AASHTO) 90%
- Maximum % passing 75 micron sieve 40%

The backfill material should be placed and compacted in layers not exceeding 300mm of compacted thickness, with each layer being compacted to the required density at the optimum moisture content. Backfill material must be placed equally on each side. Each layer must be compacted to specified density before adding the next layer. Care must be taken to ensure that no more than one layer difference each side of the structure.

Controlling the symmetry of the structure is another important thing during backfill operation, by control of the backfill operation. As a general rule, no deflection in any direction greater than 2% from original shape should be allowed during the backfill operation (Installation & Backfilling Standards for Armco Structures).

2.9 Vertical Deflection

Corrugated steel pipes functions structurally as a flexible ring which is supported by and interact with the compacted surrounding soil. The soil constructed around the pipe is thus an integral part of the structural system. Therefore it is important to ensure that the soils structure or backfill is made up of acceptable material and well constructed. Typical vertical deflection pattern is shown in figure 2.7 (Ref. : Installation & Backfilling Standards for Armco Structures) .

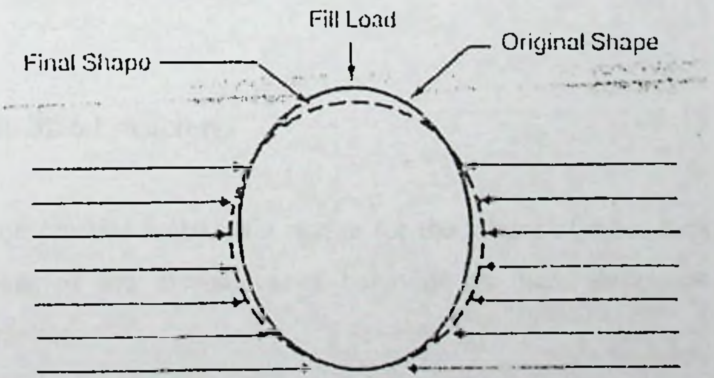


Figure 2.7 : Typical Vertical Deflection

Fill material around the structure should be placed alternatively in layers 150 or 300mm thick on both sides. Pipe – arches require that the backfill at the corners (sides) of the best material, and be especially well compacted.

The above book discuss several points we have to consider when erection of the Metal Structure.

- Placing backfill around structure
- Shape Control
- Vertical Deflection
- Construction Loadings

2.10 End Protection

This is the case with any water carrying structure, the compacted fill material around an Armco structure has to be adequately protected against erosion. This protection can take the form of concrete headwall and wing walls, stone pitching, gabion mattresses or concrete slab protection of the embankment.

The Standard concrete ring beam recommended for Armco Structures should form an integral part of the end protection of a structure. There is no substitute for adequate end protection. Which ensures that compacted fill will not be damaged by erosion or flooding.

2.11 Failures in Metal structures

Bad construction practice is the main reason for the failure of those structures. The lack of understanding of the mechanics of behavior of these structures promotes bad construction practice.

Common forms of failures are:

- Buckling of the conduit wall.
- Bolt hole tears.
- Bearing failures at longitudinal seams.
- Excessive deformation of the conduit cross section'
- Lifting of the invert

- Lifting of the pipe ends.
- Distortion of the pipe ends.
- Collapse of the structure.

2.11.1. Buckling of the conduit wall

Buckling can be a result of a local buckling in which the metallic shell buckles in to a large number of waves, each of relatively small length. Buckling can occur in the compression zone of the wall section. When the conduit wall undergoes large bending deformations, this can be usually take place in conduit segments of relatively small radius of curvature.

2.11.2. Bolt hole tears

Bolt hole tears are caused by excessive bending of the plates, and that the tendency to develop the cracks is directly related to the tension in bolts.

2.11.3. Bearing failure at longitudinal seams

Bearing failure at longitudinal seams can take place due to the yielding of the conduit wall directly under the bolts. This type of failure takes place under excessive conduit wall thrust.

2.11.4. Excessive deformation of conduit cross section

This is the most common from of failure in metal structures. This result in poorly compacted backfill, backfill containing large quantities of clay or organic matter and well compacted and good quality backfill not extending on either side of the conduit.

Excessive pipe deformation do not always develop after the structure has been built. It can deform during the initial stages of the backfilling operation.

2.11.5. Collapse of the structure

This is the most dramatic failure in these type of structures. The failure of metal underpasses is more common than other type of underpass (Design and construction of Soil steel Bridges, George Abdel-Sayed). The failure of the metal underpass could have been avoided by careful construction, using good – quality backfill and employing good construction practice.

Factors that lead to the collapse of metal structure are as follows:

- Use of poor quality soil,(containing large quantities of clay and organic matter, in the backfill)
- Compaction of backfill in very thick layers.
- Compaction of backfill in cold weather.

2.11.6 Remedial measures

The various methods that can be used as follows (Design and construction of Soil steel Bridges, George Abdel-Sayed).

- Temporary props.
- Partial concreting inside conduit.
- Internal grouting.
- Shortcreting.
- Partial concreting outside conduit.

Chapter 3

3.0 FIELD SURVEY

3.1 Introduction

This chapter describes the observations made during field survey. It describes detail description of the backfilling, construction of foundation, installation of metal structure / box culvert, backfilling, shape control, end treatment and the failures in the site.

3.2 Method and work Procedure

3.2.1 Metal Structures

3.2.1.1 Filling and Excavation

Filling and excavation width vary with the type of soil.

a) In Embankment Fill – Firm Foundation

Embankment filling & compaction will be carried out in layers. In fill sections, before excavation is begin, the fill shall be constructed for a distance of 3 span / diameter and on each side of the metal structure to a minimum height of 25% of the vertical dimension of the metal structures.

b) In Embankment Fill – Soft Foundation

The Engineer will be required to issue his instruction. Soft ground treatment works will be carried out in accordance with the Engineer's instructions.

c) In Cut – Firm Foundation (Soil)

The width of the excavation shall be 2m minimum each side of the structure.

d) In Cut – Rock Foundation

Excavation by blasting in accordance with the blasting pattern.

e) In Cut – Soft Foundation

Limits of the excavation shall be 2m minimum each side of the structure.

3.2.1.2. Foundation Preparation

- MAUP and HES structures shall be placed on a uniform stable earth or granular foundation.
- HPA structures are founded on reinforced concrete footings.

3.2.1.3. Camber at Installation

Cambering shall not be executed for the structures with concrete components (Thrust beams / footings) and in such cases foundation shall be constructed so as to avoid settlements. That is cambering shall be applicable for MAUP structures only.

3.2.1.4. Erection of Structures

MAUP structures shall be assembled in 4 stages. i.e. bottom, corner, side and top. HPA and HES structures can be assembled in 3 stages. i.e. Side, corner and top. In every type of structure plate assembling shall be proceeded at one side and the other side alternatively.

3.2.1.5. Backfilling

Structural backfill material specified in specifications shall be used as backfill material. The main deciding factor for selecting backfilling material shall be the bearing pressure of the material compacted to specified density.
Placing of backfill shall be carried out equally on both sides of the structure, in layers of compacted thickness of 300mm each layer shall be compacted to specified density before placing the next layer.

3.2.1.6 Shape Control

This refers to controlling the shape and symmetry of structure during backfilling by control of the backfill operation. Care must be taken while installing the structure.

3.2.1.7 End Treatment

Reinforced concrete shall be provided at the ends of the structure. Protection to the embankment shall be provided by concrete headwall and wing walls or gabion mattress protection.

3.2.2 Box Culvert

3.2.2.1 Filling and Excavation

Filling and excavation width and the depth vary with the type of soil.

a) In Embankment Fill – Firm Foundation

   The filling distance and height will vary with the structural dimentions and existing elevation of the location.

b) In Embankment Fill – Soft Foundation

   Soft ground treatment works will be carried out in accordance with the instructions.

c) In Cut – Firm Foundation (Soil)

   For Box culverts in soil foundation, additional excavation of 75mm below the bottom level of the culvert is needed for blinding layer.

d) In Cut – Rock Foundation

   Excavate by blasting in accordance with the blasting pattern.

e) In Cut – Soft Foundation

The excavated area will be backfilled with the suitable material and compacted up to the 300mm above the bottom of the box culvert.

3.2.2.2. Construction of Box Culverts

Formwork – class 1 will be fixed for placing concrete for base slab of box culvert. The formwork of the base slab will be removed 25hrs after concreting. Curing of all exposed concrete surface will be carried out for 7 days.

3.3. Failures in Metal Underpasses

Southern Transport Development project begins at mid of year 2002. But there were vide variety types of failures can be observed in the site.

Figure 3.1 : Husker walls are cracked.

Figure 3.2 : Weeds comes through the plates of Metal Structure.

Figure 3.3 : Plates are Corroded..

Most recently Contractor of the Southern Transport Development Project started to undergo some rectification methods.

3.4 Summary

To ensure that the steps of the procedure are fully complied with during the activities such as setting out, excavation, embankment filling, bedding, erection and backfilling etc. certain inspections and verifications will be carried out as described in the above procedures.

Chapter 4

4.0 Design of Metal Structure

4.1 Introduction

The designs are carried out to American Standards / Practices as listed below. Traffic loading has been calculated according to BS5400 Part 2, HA & HB 30 units as requested by Road Development Authority.

Design Calculations are in accordance with following references:

1. Handbook of Steel Drainage & Highway Construction Products (American Iron & Steel Institute)
2. ASTM Designation A 761/A 761M-98 – Corrugated Steel Structural Plate , Zinc – Coated, for field – Bolted Pipe, Pipe – Arches and Arches.
3. ASTM Designation A 796/A 796M-01 – Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe Arches and Arches for Storm and Sanitary Sewers and other Buried Applications.
4. Standard Specifications for Highway Bridges (AASHTO) – Section 12
5. British Standard for Steel , Concrete and Composite Bridges – BS5400 Part 2 : 1978.

4.2 Description of Loads on the Metal Structure

4.2.1 Dead Load

The following unit weight of materials were used for the design

- Compacted Density of Asphalt Concrete Surfacing – 24.030 kN/m^3
- Compacted fill (Soil) – 19.0 kN/m^3

4.2.2. Live Load HB Loading

As per BS5400;
16 wheels, each 75kN (30units of HB) as shown in Figure 11 of BS5400 with innermost axels 6m apart shall be considered. Accordingly the square contact area shall be 261mm a side.

4.3. Design Calculation as per AASHTO (For HPA 74N)

4.3.1 Description of the Proposed Structure

(a).	Structure Type	HPA 74N/ Armco MP152S
(b).	Dimensions	Max. span 10.58m x rise 5.42m
(c).	Top Radius	7.32m
(d)	Corner Radius	1.65m
(e)	Corrugation	152mm x 50.8mm
(f)	Specified thickness – Top Arc	6.32mm (Armco Nominal Thickness 6.35mm)
(g)	Specified thickness - Remainder	6.00mm (Armco Nominal Thickness 6.00mm)

Table 4.1 : Description of the Proposed Structure

4.3.2 Outline Drawing of the Proposed Structure

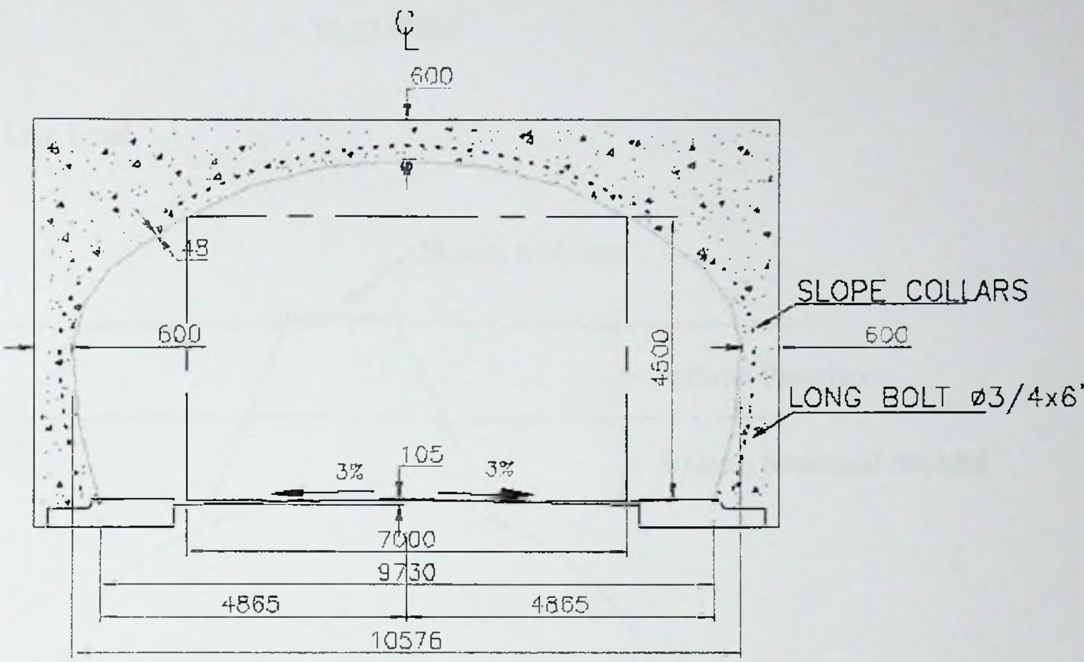
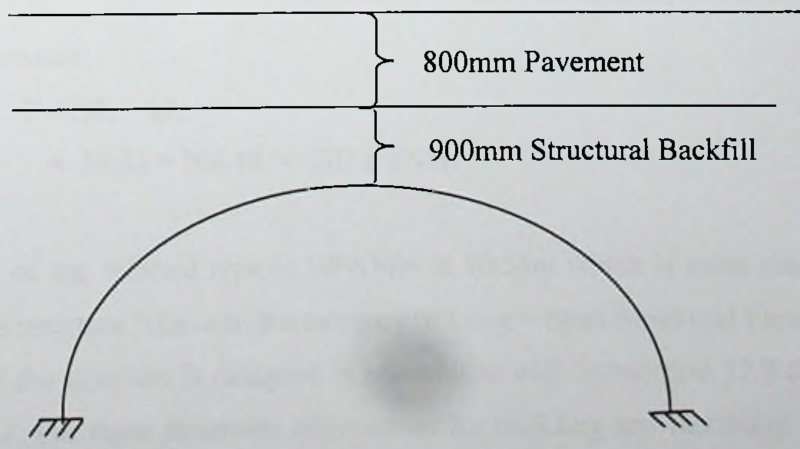


Figure 4.1 : Concrete Outline of the HPA74N

4.3.3. Design Pressure

The Design Pressure shall be;

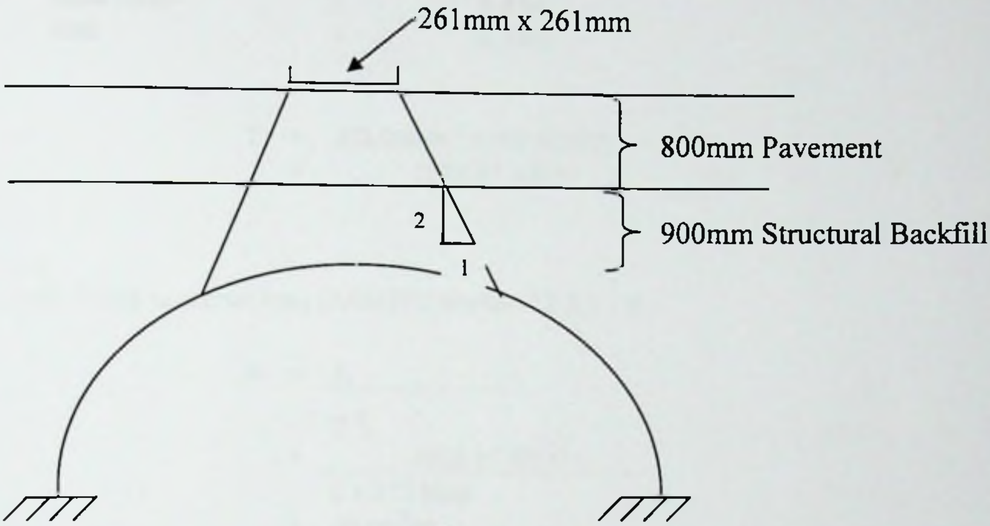
P = DL + LL (from AASHTO Section 3.8.2)



Dead Load (DL) $= (0.9 \times 19) + (0.8 \times 24.03)$

$= 36.32\ kN/m^2$

Live Load



Live Load (LL) $= \dfrac{75 \times 9.81}{(0.261+0.7+0.7) \times (0.261+0.7+0.7)}\ kN/m^2$

$= 266.68\ kN/m^2$

Design Pressure

$P = DL + LL$

$= 36.32 + 266.68 = 303.0\ kN/m^2$

The span of the selected type is HPA74N is 10.58m which is more than 6.4m and hence this structure falls in to the category of Long – Span Structural Plate Structures. Therefore the structure is designed in accordance with Subsection 12.7 of AASHTO Section 12. For these structures requirement for buckling and flexibility shall not be applied.

According to AASHTO Section 12.1.4.2.

The thrust in the wall is ; $T = P \times S/2$

For HPA 74N

Maximum span	=	10.58m	(>6.4m i.e. long span structural plate structures) (ASTM 796 Section 5)
Base span	=	9.81m	
Rise	=	5.54m	

$$T = 303.0\text{kN/m}^2 \times (10.58\text{m}/2)$$
$$= 1602.87 \text{ kN/m}$$

Wall Cross sectional Area (AASHTO Section 12.3.1.) is ;

$$A = \frac{T_L}{\varphi f_y}$$
$$= \frac{1602.87 \text{ kN/m}}{2 \times 310 \text{ Mpa}}$$
$$= 26 \text{ cm}^2/\text{m}$$

Required wall area $= 26.0 \text{ cm}^2/\text{m}$

Area corresponding to the thickness of proposed structure is 74.631 cm^2/m. (Thickness 6.00mm)

$A = 26.0 < 74.631 \text{ cm}^2/\text{m}$
Hence ok.

Check as per AASHTO Section 12.7

I.Table 12..7.2.A Minimum Requirement for Long – Span Structures with Acceptable Special Features.

In HES 74N Top Radius 7.32m = 24.01 ft

Refer Table 12.7.2.A ;

Top arc thickness = 0.249 in = 6.32mm

Specified thickness of Top – Arc of proposed structure = 6.32mm

Hence OK.

II As per Table 12.7.2.A.

The Minimum cover is 4.0ft

(i.e. 1.22m for 23 – 25 ft Top Radius & 0.249in Steel thickness)

III According to AASHTO Geometric Limits

A . Maximum plate radius = 7.32m = 24. ft < 25ft

Hence OK.

B. Maximum central angle of Top arc = 80^0

Central Angle of top arc of proposed structure = 80^0

Hence OK.

C. Minimum Ratio , Top Arc Radius to Side Arc Radius = 2

For Proposed structure

Top Arc Radius = 7.32m

Side Arc Radius = 1.65m

Therefore Ratio for Proposed Structure = 4.436 > 2

Hence OK.

Therefore dimensions of the proposed structure satisfied

Chapter 5

Analysis of Box Culvert, Results & Design

5.1 Introduction

Analysis of Box Culvert was performed in SAP 2000. The box culvert should be designed considering the following issues.

- It should be able to discharge the volume expected during a design flood.
- The structure of drainage culvert should be designed to be stable against the dead , superimposed dead, live and earth pressure.

5.2 Load Cases

Following load cases are to be taken in to account(since culvert is used as underpass)

1. Vehicles at Top – No vehicles at Bottom
   1.1 HA only
   1.2 HA & HB only
2. No Vehicles at Top – No vehicles at Bottom
   2.1 No HA or HB
3. No Vehicles at top – vehicles at bottom
   3.1 HA only
   3.2 HA & HB only
4. Vehicles at Top & Bottom both
   4.1 HA only in top & bottom both
   4.2 HA in top & bottom & HB in bottom only
   4.3 HA & HB in top & bottom both
   4.4 HA in top & bottom & HB in top only

5.3 Loads on the Box Culvert

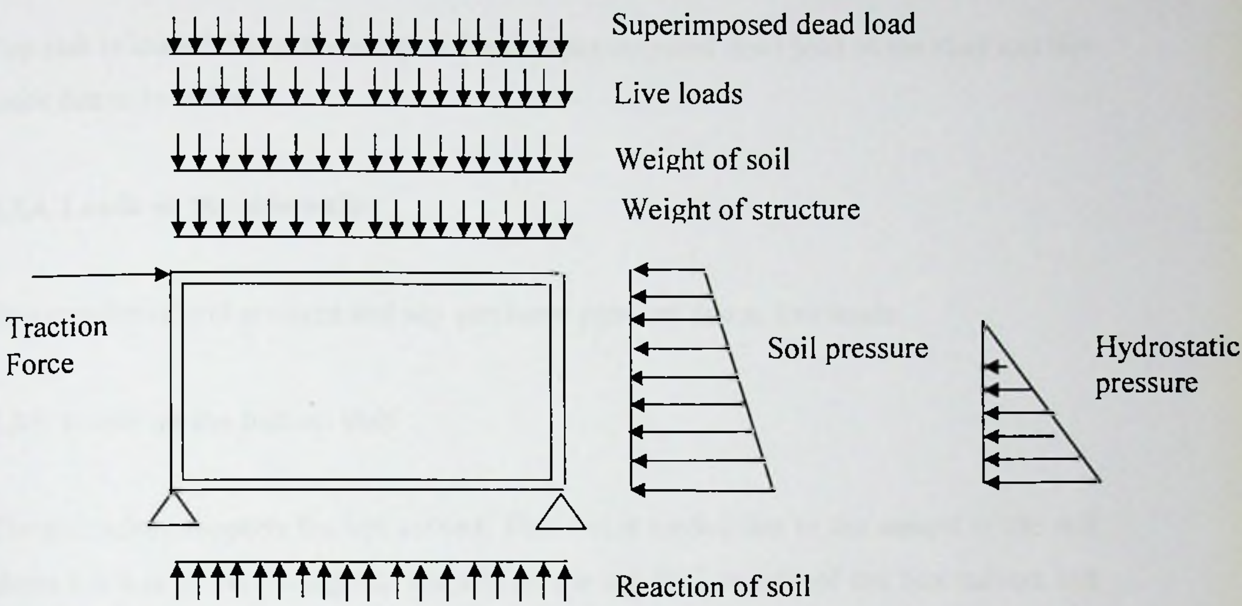


Figure 5.1 : Loads on the Box Culvert

5.3.1 Loads due to soil

Depending on the level of the stream, the box culvert can be either at the road level or buried. If it is buried, there will be soil on all four sides. Thus the following loads will act due to the soil :

- The weight of the soil between the top slab and the road level acting on the top slab.
- The soil pressure acting on the sides of the box culvert.
- If there is soil on the top slab, the soil loads will be transferred on to the side walls.

5.3.2 Live Loads

Live loads are generally due to vehicles traveling on the road. Live loads are calculated according to BS5400.

5.3.3. Loads on the top slab.

Top slab is loaded due to the weight of soil, super-imposed dead load of the road and live loads due to vehicles.

5.3.4. Loads on the side walls

This consists of soil pressure and any surcharge pressure due to live loads.

5.3.5. Loads on the bottom slab

The soil below supports the box culvert. This soil is loaded due to the weight of the soil above the box culvert, weight of the soil on the top slab, weight of the box culvert and live load on the box culvert. The average upward pressure is assumed on the bottom slab. This pressure is equal to all the loads divided by the bottom slab area.

5.3.6 Horizontal Live load due to traction

The structure shall be designed to resist the traction forces. Traction force shall be applied perpendicular to the walls of the box culvert. Traction force was calculated as in accordance with section 6.6 of BS5400 Part 2.

5.3.7. Hydrostatic Pressure

The effect of hydrostatic pressure must be taken in to account in the design of box culverts. (either the selected structure is vehicular culvert). Because due to heavy rain if water table increases to the high flood level in that area it will automatically generate high hydrostatic pressure.

5.4 Modeling of Box Culvert

Analysis of box culvert was performed using 2D shell elements of appropriate thickness. Soil was modeled as springs. Then the support conditions are taken as simply supported at its two ends.

Depending on the SPTN values of each soil type spring constants are calculated and tabulated below.

Depth (measured from the culvert top level) (in mm)	Spring Constants
2675	8,000
2140	16,000
Below 2140	24,000

5.4.1 Load Calculation

5.4.1.1 Dead Load Calculation

Dead load may be calculated from SAP2000 finite element software automatically.

5.4.1.2. Live load calculation

HA Loading

UDL shall be taken as 30kN per linear meter of notional lane.

HA loading for the Top Slab = 30kN/m

HA loading for the Bottom Slab = 30kN/m

HB Loading

HB loading for the top slab - 30 units each 75kN

HB loading for the bottom slab - 20 units each 50kN

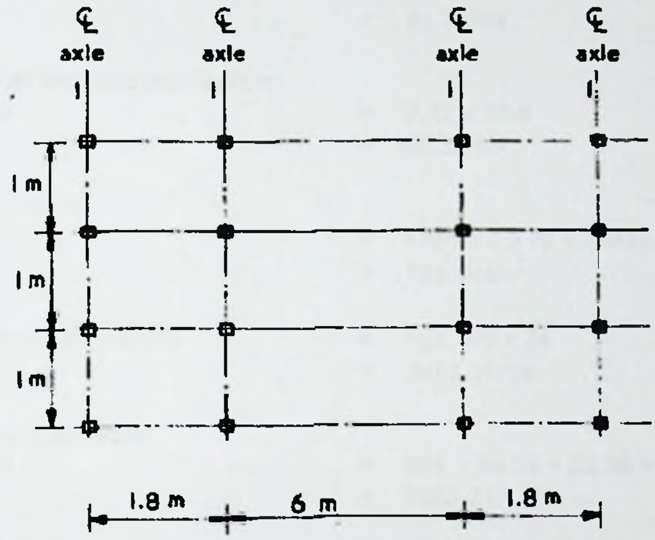


Figure 5.2 : HB Vehicle Wheel Arrangement

5.4.1.3. Super Imposed Dead Load

Assume 60mm surfacing

Compacted Density of Asphalt Concrete = $2450 kg/m^3$

Loading from the surfacing = 2450 x 0.06 x 5.35 x 9.81

= 7.72 kN/m

5.4.1.4. Reaction from Soil Calculation

Load Case 1.1

Total HA load in top slab	=	30 x 10.8
	=	324 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	103.148m^3
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	324 + 83.38 + 83.38 + 2475.55
	=	2966.31 kN
Reaction from soil	=	274.66 kN/m

Load Case 1.2

Total HA load in top slab	=	30 x 10.8
	=	324 kN
Total HB load in top slab	=	75 x 4 x 4
	=	1200 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	103.148m^3
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	324 + 1200 + 83.38 + 83.38 + 2475.55

	=	4166.31 kN
Reaction from soil	=	385.77 kN/m

Load Case 2.1

Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	103.148m^3
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	83.38 + 83.38 + 2475.55
	=	2642.31 kN
Reaction from soil	=	244.66 kN/m

Load Case 3.1

Total HA load in bottom slab	=	30 x 10.8
	=	324 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	103.148m^3
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	324 + 83.38 + 83.38 + 2475.55
	=	2966.31 kN
Reaction from soil	=	274.66 kN/m

Load Case 3.2

Total HA load in bottom slab	=	30 x 10.8
	=	324 kN
Total HB load in bottom slab	=	50 x 4 x 4
	=	1200 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	103.148m^3
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	324 + 1200 + 83.38 + 83.38 + 2475.55
	=	3766.31 kN
Reaction from soil	=	348.73 kN/m

Load Case 4.1

Total HA load in top & bottom slab	=	30 x 10.8 x 2
	=	648 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	103.148m^3
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	648 + 83.38 + 83.38 + 2475.55
	=	3290.30 kN
Reaction from soil	=	304.66 kN/m

Load Case 4.2

Total HA load in top & bottom slab	=	30 x 10.8 x 2
	=	648 kN
Total HB load in bottom slab	=	50 x 4 x 4
	=	1200 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	$103.148m^3$
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	648 + 1200 + 83.38 + 83.38 + 2475.55
	=	4090.30 kN
Reaction from soil	=	378.73 kN/m

Load Case 4.3

Total HA load in top & bottom slab	=	30 x 10.8 x 2
	=	648 kN
Total HB load in top slab	=	75 x 4 x 4
	=	1200 kN
Total HB load in bottom slab	=	50 x 4 x 4
	=	800 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	$103.148m^3$

Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	648 + 1200 + 800 + 83.38 + 83.38 + 2475.55
	=	5290.33 kN
Reaction from soil	=	489.85 kN/m

Load Case 4.4

Total HA load in top & bottom slab	=	30 x 10.8 x 2
	=	648 kN
Total HB load in top slab	=	75 x 4 x 4
	=	1200 kN
Super imposed load top slab	=	7.72 x 10.8
	=	83.38 kN
Super imposed load bottom slab	=	7.72 x 10.8
	=	83.38 kN
Volume of concrete	=	{10.8 x (0.75 + 0.60) x 5.35} + 9 4.7 x 0.5 x 5.35 x 2)
	=	103.148m^3
Weight of concrete	=	103.148 x 24
	=	2475.55 kN
Total downward load	=	648 + 1200 + 83.38 + 83.38 + 2475.55
	=	4490.30 kN
Reaction from soil	=	415.77 kN/m

5.4.1.5 Lateral earth pressure calculation

Take Surcharge as 10kN/m^2

Take Unclassified Soil

γ = 18 kN/m3

φ = 30 dgrees

Consider at Rest condition

k_0 = (1 - Sin φ)

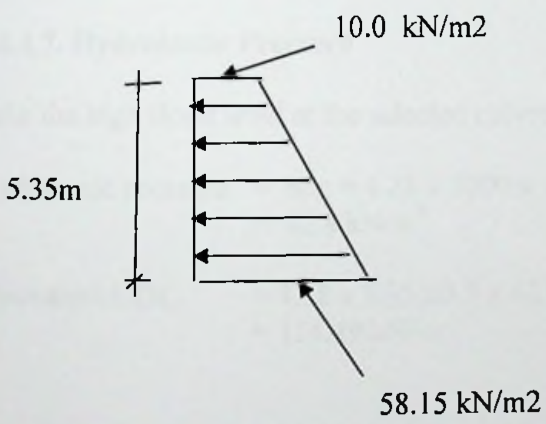
= 0.5

Over-burden at top = 10kN/m2

Over-burden at 5350mm depth = 10 + $k_0\gamma H$ = 10 + 0.5 x 18 x 5.35

= 58.15 kN/m2

Pressure Distribution



5.4.1.6. Traction force

BS5400 Part 2 Section 6.6 states that

For HA loading :

Traction force = 8 x 10.8 +200 (but minimum 700kN)

Traction force from HA vehicle = 286.4 kN

For HB loading

Traction force for 30units of HB vehicle = 75 x 25% x 8
= 150 kN

Traction force for 20units of HB vehicle = 50 x 25% x 8
= 100 kN

From the above calculations Traction force for HA vehicle > HB vehicle

Apply traction force of 286.4kN from HA vehicle at the center of the notional lane.

5.4.1.7. Hydrostatic Pressure

Take the high flood level at the selected culvert area as 4.28m

Hydrostatic pressure = $h\rho g$ = 4.28 x 1000 x 10 / 1000
= 42.8 kN/m^2

Equivalent UDL = 42.8 x 5.35 x0.5 x 42.8/42.8
= 114.49 kN/m

5.5 Concrete Outline Drawing of Box Culvert

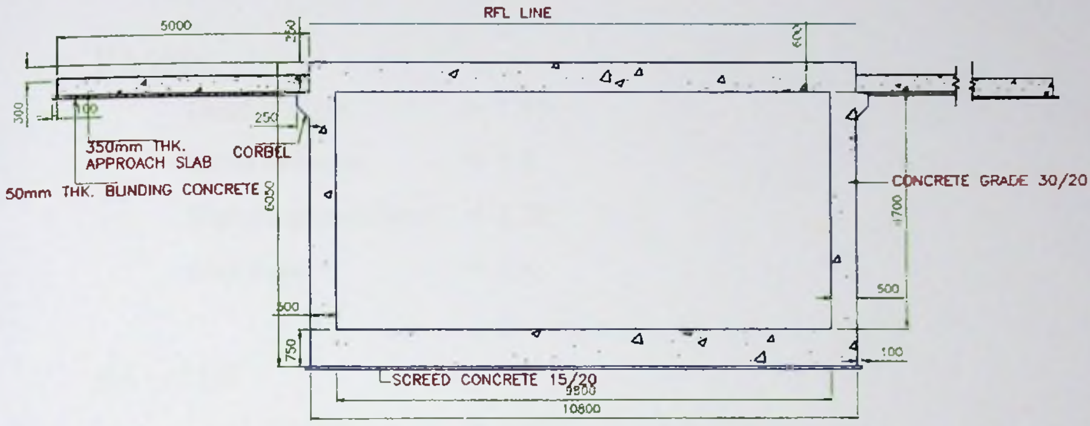


Figure 5.3 : Concrete Outline of Box Culvert

5.6. SAP2000 model of the Box Culvert

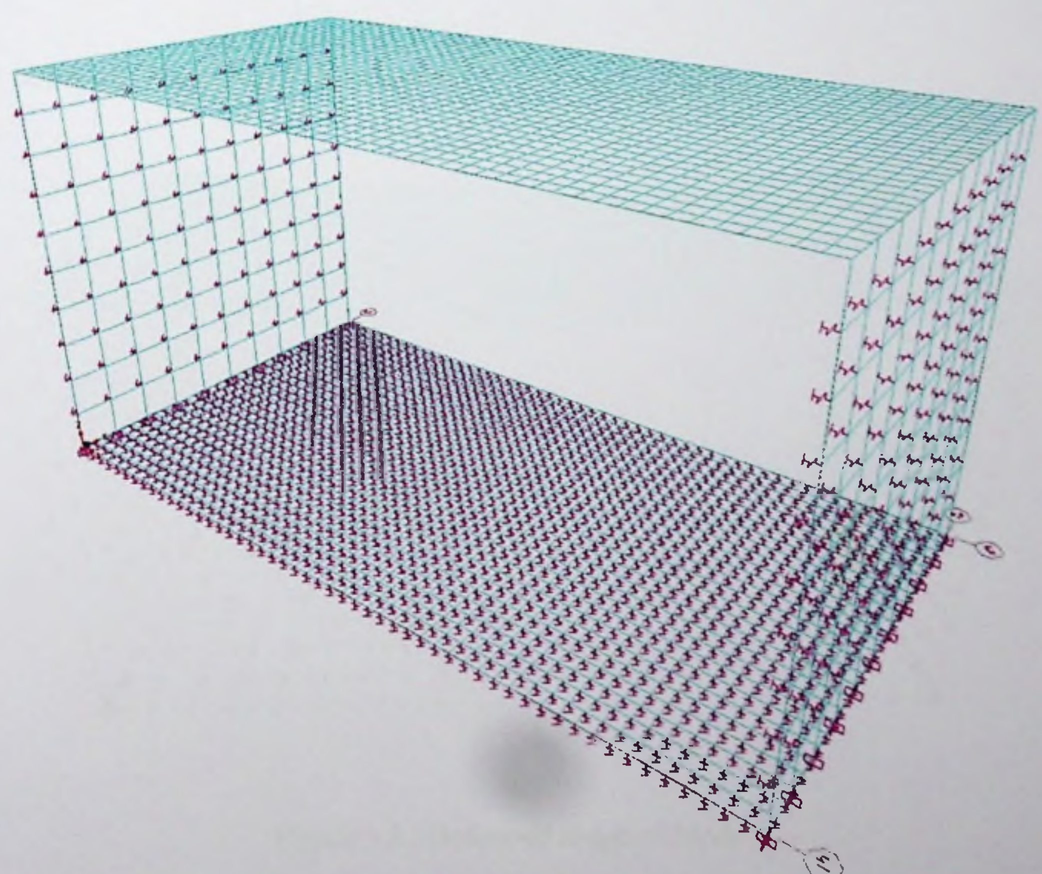


Figure 5.4 : Model of Box Culvert

5.7 Load Combinations

HA Only

Dead Weight	=	1.15
Earth Pressure	=	1.5
Superimposed Dead	=	1.75
Live Load	=	1.5

HA and HB

Dead Weight	=	1.15
Earth Pressure	=	1.5
Superimposed Dead	=	1.75
Live Load	=	1.3

5.8 Deformed Shape for Load Case 4.3

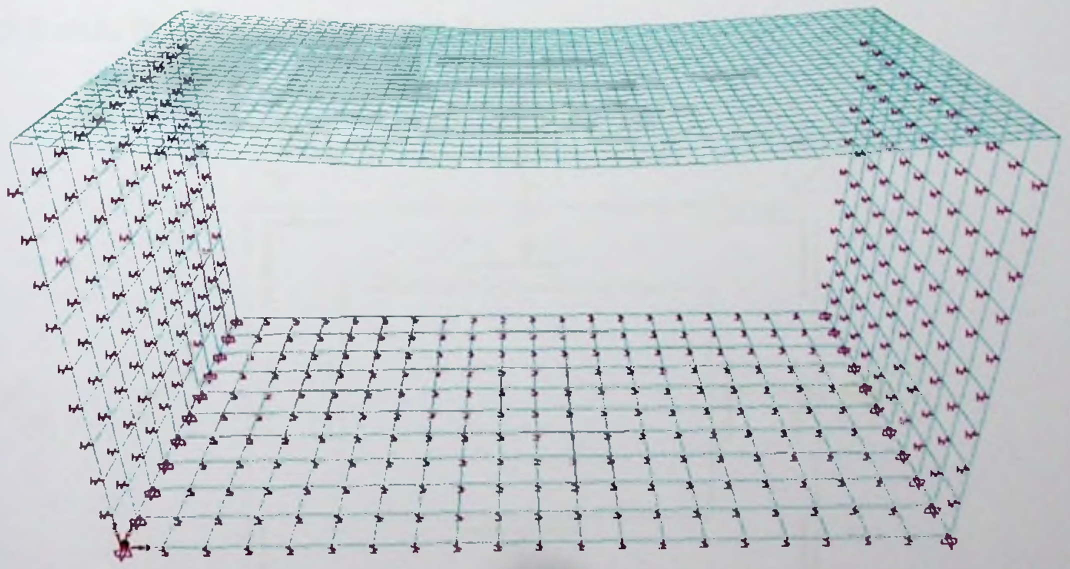


Figure 5.5 : Deformed shape of Mode 1

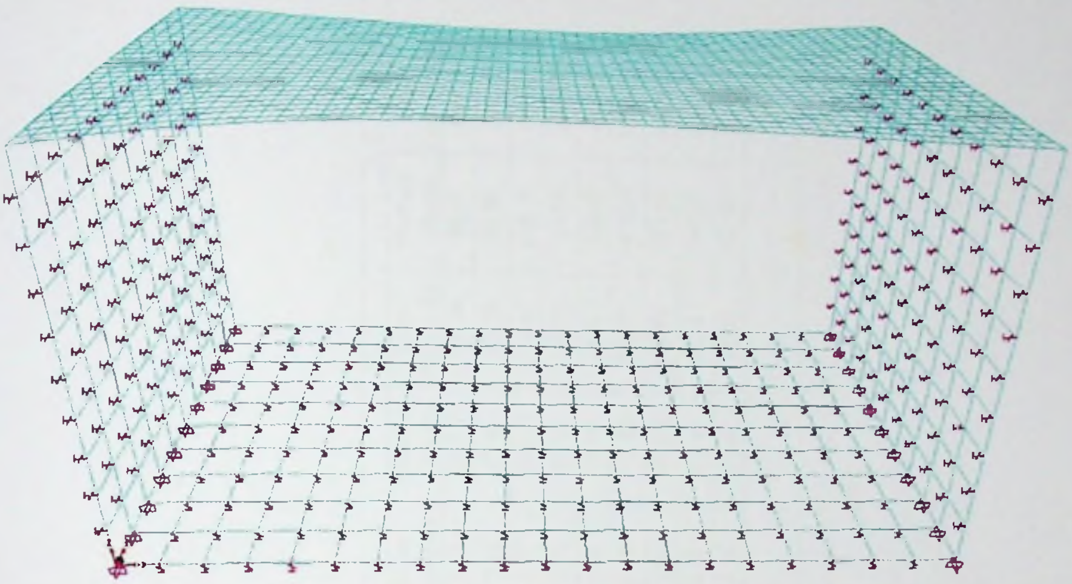
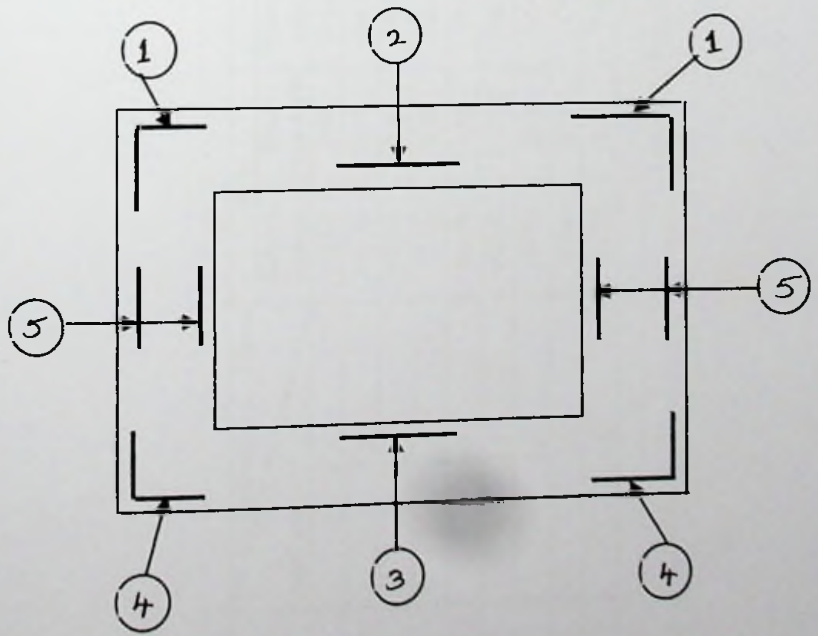


Figure 5.6 : Deformed shape for Mode 2

5.9 Results from the SAP2000 Modeling



Moments & Forces / Load Case	At 1		At 2		At 3		At 4		At 5	
	Moment kNm/m	Shear Force kN/m	Moment kNm/m	Shear Force kN/m	Moment kNm/m	Shear Force kN/m	Moment kNm/m	Shear Force kN/m	Moment kNm/m	Shear Force kN/m
1.1	141.50	3.97	212.39	38.56	159.79	0.17	56.74	108.34	98.91	686.71
1.2	241.30	11.00	244.01	57.96	86.91	0.14	41.46	90.28	93.88	368.16
2.1	388.65	0.80	179.50	47.30	162.70	0.27	124.75	117.56	205.24	951.77
3.1	17.53	3.32	358.85	82.67	133.14	0.18	51.15	184.27	14.55	951.22
3.2	124.38	0.52	38.50	25.21	31.04	0.07	18.79	9.51	302.96	362.79
4.1	450.50	1047.50	488.35	100.68	133.14	0.18	113.63	349.77	413.54	1050.75
4.2	164.53	28.35	53.45	2.99	14.56	0.05	560.08	196.67	373.54	448.49
4.3	130.44	48.81	4.75	2.66	9.94	0.07	570.46	203.41	956.56	310.46
4.4	332.98	5.12	9.69	1.49	86.91	0.14	41.46	90.28	93.88	451.02

Table 5.1 :Results from the SAP2000 Modeling

Chapter 6

6.0 COST ANALYSIS

6.1 Introduction

One of the main objectives of this project is to evaluate the cost effectiveness of metal structure when compared with box culvert, to select most appropriate structure type for a site under consideration. This chapter provides guidelines for the establishment of basic cost to compare the alternatives.

The basic cost used here is taken form currently used for estimating process at Road Development Authority.

6.2 Cost Estimating Process

The process stated below is developed for estimating the underpass cost after the completion of the preliminary design. Cost for all other items including but not limited to the following are excluded from the cost provided in this chapter.

6.3 Cost Analysis of Metal Structure

The basic structural items of the HPA74N metal structure was found to be as follows.

Bill of Material for HPA74N Metal Structure

Item No.	Description	Units	Total Nos.	Width (m)	Length (m)	Height (m)	Total Quantity
1	HPA74N Metal Plates	Nos.	119	-	-	-	119
2	Bolts (3/4" x 3")	Nos.	5219	-	-	-	5219
3	Unclassified Excavation	m^3	1	14.58	20.13	7.6	2231.0
4	Structural Backfill for Metal Structure	m^3	1	10.58	20.13	0.6	128.00
			1	3.00	20.13	4.4	266.0
5	Foundation	m^3	2	1.0	20.13	0.45	18.12
			2	0.2	20.13	(0.45+0.3)/2	3.02
			2	0.6	20.13	0.3	7.25

Cost Calculation

The basic cost presently used in Road Development Authority was used.

Cost of Concrete

1. Grade 30 concrete per m^3 with placing — Rs. 13,000.00
2. Reinforcing Steel (100kg per m^3) — Rs. 15,000.00

Concrete cost with r/f per m^3 — Rs. 28,000.00

Item	Unit	Rate (Rs.)	Quantity	Amount
HPA74N Metal Plates	Nos.	291,500.00	119	34,688,500.00
Bolts (3/4" x 3")	Nos.	15.00	5219	78,285.00
Unclassified Excavation	m^3	533.00	2231.0	1,189,123.00
Structural Backfill for Metal Structure	m^3	8342.62	394.0	3,286,992.28
Foundation	m^3	28,000.00	28.39	794,920.00
Total Cost				40,037,820.28

6.4. Cost Analysis of Box Culvert

The basic structural items of the reinforced concrete box underpass at 09+373 were found to be as follows.

Bill of Material for Reinforced Concrete Box Underpass

Item No.	Description	Units	Cross Sectional Area (m^2)	Width (m)	Length (m)	Height (m)	Total Quantity
1	Grade 30 Structural Concrete	m^3	-	-	-	-	437.119
2	Reinforcement	Tones	-	-	-	-	32.365
2	Unclassified Excavation	m^3	1	14.30	20.13	7.0	2015.0
3	Formwork	m^2	-	-	20.13	6.05	440.85
				-	20.13	6.05	
				-	20.13	9.8	

Cost Calculation

Item	Unit	Rate (Rs.)	Quantity	Amount
Grade 30 Structural Concrete	m^3	13,000.00	437.119	5,682,547.00
Reinforcement	Tonnes	150,000.00	32.365	4,854,750.00
Unclassified Excavation	m^3	533.00	2015.0	1,073,995.00
Formwork	m^2	2033.21	440.85	896,334.53
Total Cost				12,507,632.63

Note that the above calculated cost is very basic cost not in the actual cost involving the construction of metal underpasses and box culvert. For the above cost calculation the cost of construction of wing wall, mobilization,

6.5 Cost Saving

According to the above basic cost calculation the average cost saving for the structural part as follows.

Cost saving	=	Rs. 40,037,820.28- 12,507,632.63
	=	Rs. 27,530,187.65
% of cost saving	=	Rs. 27,530,187.65 / 40,037,820.28x 100%
	=	68.7%

Chapter 7

7.0 CONCLUSIONS AND RECOMMENDATION

According to the cost calculation it is shown that the 74% of cost saving is there by using box underpass.

By using box underpass instead of metal underpass Sri Lanka can save billions of money from Southern Transport Development Project.

Another useful source of information is the durability of metal underpasses. Sacrificial cathodic protection is the effective method of protecting the metallic shell against corrosion. In this protective system the steel plate is connected through an electrical conductor to a sacrificial zinc or aluminium, plate which acts as anode and corrodes, there by preventing the steel plate from corrosion.

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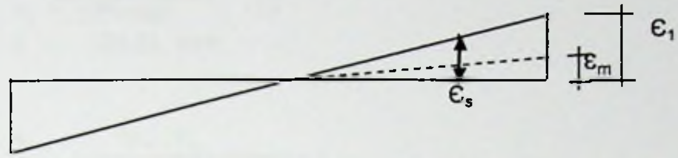
APPENDIX -A

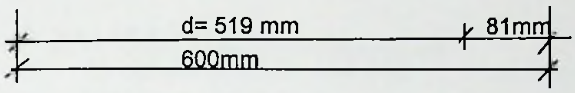
Reference	Calculation	Output
	<u>**DESIGN OF DECK SLAB (TOP SLAB)**</u>	
	Bending moment is critiacl in Load case 4.1. So that consider the loading case 4.1 for reinforcement design.	
	Design main reinforcement for mid span of the slab.	
	Thickness of the slab = 600 mm	
Part 4: 1990 4.2.3	Design bending moment, M design at mid span = 541 kNm/m	M = 541 kNm/m
	Assume severe environment condition, Cover = 65 mm	Cover = 65 mm
	Diametre of main reinforcement = 32 mm	
	Effective depth, d = 600-65-32/2 = 519 mm	d = 519 mm
BS 5400 Part 4: 1990 5.3.2.3	$M = (0.87f_y)A_s z$ equation 1 $z = (1 - 1.1f_yA_s/ f_{cu}bd)\ d$ equation 5 from these two equations $z = 0.5d\ [1+ (1-5M/f_{cu}bd^2)^{1/2}\ ]$ $z = 0.5d\ [1+ (1-5\text{x}541\text{x}10^6/30\text{x}1000\text{x}523^2)^{1/2}\ ]$ = 0.908 d < b d Hence o.k Z = 0.908 d = 0.908x519 = 471 mm	
equation 1	**Main reinforcement** $A_s = M / 0.87f_yz$ $= 541\text{x}10^6 / 0.87\text{x}460\text{x}471$ = 2868 mm^2/m Use T 32 @ 275 (As = 2925 mm^2)	$A_{s\ req}$ = 2868 mm^2/m $A_{s\ pro}$ = 2925 mm^2/m
BS 5400 Part 4: 1990 5.8.4.1	**Check for minimum reinforcement required for cantilever slab** Check for minimum reinforcement $100A_s / b_ad$ = 100x2925/(1000x519) = 0.56 > 0.15 Hence o.k	Main reinforcement T 32 @ 275
BS 5400 Part 4: 1990 5.8.4.2	(b) <u>DESIGN OF SECONDARY REINFORCEMENT (TOP SLAB)</u> Requirement for secondary reinforcement $(0.12/100)b_td$ = (0.12/100)x1000x519 = 623 mm^2/m Use T **12** @ **175** (As = 646 mm^2)	Secondary reinforcement T **12** @ **175**

Reference	Calculation	Output

Serviceasibility Crack check

At center of bottom slab maximum service moment = 450.5 kNm





$$\frac{x}{d} = -\alpha e\rho + \sqrt{\alpha e\rho(2+\alpha e\rho)}$$

$$\alpha e = \frac{Es}{Ec} = 14.2857$$

$$\rho = \frac{A_s}{bd} = \frac{2925}{10^3 \times 519} = 0.005$$

$$\alpha e\rho = 14.286 \times 0.005$$
$$= 0.07143$$

$$\frac{x}{d} = 0.31$$

$$x = 162.56 \text{ mm}$$

$$\frac{z}{d} = 1 - \frac{1}{3}\left(\frac{x}{d}\right)$$

$$z = 464.81 \text{ mm}$$

Service stress, $f_s = \frac{M}{A_s \times Z}$

$$fs = \frac{450.5 \times 10^6}{2925 \times 464.81}$$
$$= 331.406 \text{ N/mm}^2$$

$$\epsilon_s = \frac{f_s}{E_s} = \frac{331.406}{200 \times 10^3} = 0.0017$$

$$\epsilon_1 = \left(\frac{h - x}{d - x}\right) \times \epsilon_s$$
$$= \left(\frac{600 - 162.564}{523 - 162.564}\right) \times 0.0017$$
$$= 0.002$$

$$\epsilon_m = \epsilon_1 - \frac{b(h-x)^2}{3E_sA_s(d-x)}$$
$$= 0.0020336 - \frac{1000 * (600 - 162.564)^2}{3 \times 200 \times 1000 \times 2925\,(519 - 162.56)}$$
$$= 0.002$$

Reference	Calculation	Output
	Calculation of a_{cr}	
	C_{min} = 81 mm	
	$a_{cr} = \sqrt{97^2+100^2} - 16$	
	= 123.32 mm	
	$w_{cr} = \dfrac{3a_{cr}\ \varepsilon_m}{1 + 2[(a_{cr} - c_{min})/(h-x)]}$	
	$w_{cr} = \dfrac{3 \times 123.32 \times 0.002}{1+2[(123.32-81)/(600-162.56)]}$	
	= 0.19 mm < 0.2mm	
	Hence Satisfactory.	w = 0.19 <0.2mm hence satisfactory
	DESIGN MAIN REINFORCEMENT FOR HOGGING MOMENT (TOP SLAB)	
	Thickness of the slab = 600 mm	
BS5400 Part 4: 1990 4.2.3	Design bending moment, M design at mid span = 586 kNm/m	M = 586 kNm/m
	Assume severe environment condition, Cover = 65 mm	Cover = 65 mm
	Diametre of main reinforcement = 20 mm	
	Effective depth, d = 600-65-20/2 = 525 mm	d = 525 mm
BS 5400 Part 4: 1990 5.3.2.3	$M = (0.87f_y)A_sz$ equation 1	
	$z = (1 - 1.1f_yA_s/ f_{cu}bd)\ d$ equation 5 from these two equations	
	$z = 0.5d\ [1+ (1-5M/f_{cu}bd^2)^{1/2}]$	
	$z = 0.5d\ [1+ (1-5\times586\times10^6/30\times1000\times525^2)^{1/2}]$	
	= 0.90 d < 0.950 d	
	Hence o.k	
	Z = 0.90 d	
	= 0.900x525 = 473 mm	
	Main reinforcement	
equation 1	$A_s = M / 0.87f_yz$	
	$= 586\times10^6 / 0.87\times460\times473$ = 3093 mm^2/m	$A_{s\ req}$ = 3093 mm^2/m
	Use T 20 @ 100 (As = 3142 mm^2)	$A_{s\ pro}$ = 3142 mm^2/m
	Check for minimum reinforcement required for cantilever slab	
BS 5400 Part 4: 1990 5.8.4.1	Check for minimum reinforcement	
	$100A_s / b_ad$ = 100x3142/(1000x525) = 0.598 > 0.15	Main reinforcement T 20 @ 100
	Hence o.k	

Reference	Calculation	Output

<u>Serviceability Crack Check</u>

At corners maximum service moment = 488.35 kNm

From the above

$$\alpha ep = 0.071$$

$$x = 162.56 \text{ mm}$$

$$z = 464.81 \text{ mm}$$

$$fs = \frac{488.35 \times 10^6}{3142 \times 464.81}$$

$$= 334.386 \text{ N/mm}^2$$

$$\epsilon_s = \frac{f_s}{E_s} = \frac{334.386}{200 \times 10^3} = 0.0017$$

$$\epsilon_1 = \left(\frac{h - x}{d - x}\right) \times \epsilon_s$$

$$= \left(\frac{600 - 162.56}{525 - 162.56}\right) \times 0.0017$$

$$= 0.002$$

$$\varepsilon_m = \epsilon_1 - \frac{b(h-x)^2}{3E_sA_s(d-x)}$$

$$= 0.002 - \frac{1000 * (600 - 162.56)^2}{3 \times 200 \times 1000 \times 3142 (525 - 162.56)}$$

$$= 0.0017$$

Calculation of a_{cr}

$$C_{min} = 75 \text{ mm}$$

$$a_{cr} = \sqrt{75^2+50.0^2} - 10$$

$$= 80.14 \text{ mm}$$

$$w_{cr} = \frac{3a_{cr} \; \varepsilon_m}{1 + 2[(a_{cr} - c_{min})/(h-x)]}$$

$$w_{cr} = \frac{3 \times 80.14 \times 0.0017}{1+2[(80.14-75)/(600-162.56)]}$$

$$= 0.19 \text{ mm} < 0.2\text{mm}$$

Hence Satisfactory.

w = 0.19
<0.2mm
hence satisfactory

Minimum Steel : 0.18% each face , each way

$$Area = \frac{0.18}{100} \times 10^3 \times 600 = 1080 \text{ mm}^2\text{/m}$$

Use Y16 @ 175mm centers (1149 mm^2/m)

Reference	Calculation	Output
	<u>DESIGN FOR SHEAR</u>	
	Design shear force, V design = 1257 kN/m	
	Effective depth, d = 525 mm	
BS 5400: Part 4: 1990 5.3.3.1 equation 8	Design shear stress, v $= V/bd$ $= (1257\text{x}10^3)/(1000\text{x}525)$ $= 2.39$ N/mm^2 $0.75X(f_{cu})^{1/2}$ $= 0.75\text{x}(30)^{1/2}$ $= 4.108$ N/mm^2 Design shear stress, v $= 2.39 < 0.75\text{x}(f_{cu})^{1/2}$ or 4.75 N/mm^2 Hence O.K	v = 2.39 N/mm^2
BS 5400: Part 4: 1990 5.3.3.2	For uniaxial shear Allow. shear resistance ,$\xi_s v_c$ $= (0.27/\gamma_m)(100A_s/b_w d)^{1/3} (f_{cu})^{1/3} \xi_s$ Where, depth ratio, ξ_s $= (500/d)^{1/4}$ $= (500/525)^{1/4}$ $= 0.99$ or 0.7 (greater value) $\xi_s v_c$ $= (0.27/0.99)\text{x}(100\text{x}3142/1000\text{x}525)^{1/3}\text{x}(30)^{1/3}\text{x}1.25$ $= 0.89 < v = 2.39$ N/mm^2 Hence shear r/f is required	
	<u>DESIGN OF wall</u>	
	Consider load case 4.3 for design the wall of the culvert.	
	Design main reinforcement for mid span of the slab.	
	Thickness of the slab = 500 mm	
Part 4: 1990 4.2.3	Design bending moment, M design at mid span = 1148 kNm/m	M = 1148 kNm/m
	Assume severe environment condition, Cover = 45 mm	Cover = 45 mm
	Diametre of main reinforcement = 32 mm	
	Effective depth, d = 500-45-32/2 = 439 mm	d = 439 mm
BS 5400 Part 4: 1990 5.3.2.3	$M = (0.87f_y)A_s z$ equation 1 $z = (1 - 1.1f_y A_s/ f_{cu}bd)\ d$ equation 5 from these two equations $z = 0.5d\ [1+ (1-5M/f_{cu}bd^2)^{1/2}]$ $z = 0.5d\ [1+ (1-5\text{x}1148\text{x}10^6/30\text{x}1000\text{x}439^2)^{1/2}]$ $= 0.543$ d < b d Hence o.k $Z = 0.543$ d $= 0.543\text{x}439$ = 238 mm	

Reference	Calculation	Output
	Main reinforcement	
equation 1	$A_s = M / 0.87 f_y z$	
	$= 1148 \times 10^6 / 0.87 \times 460 \times 238 = 12032 \; mm^2/m$	$A_{s\,req} =$ 12032 mm^2/m
	Use T 32 @ 50 (As = 16085 mm^2)	$A_{s\,pro} =$ 16085 mm^2/m