ADDIS ABABA INSTITUTE OF TECHNOLOGY (AAiT)

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Provision of Underpass Pedestrian Crossings in Addis Ababa Railway Construction Projects

By

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A Thesis Submitted to School of Graduate Studies of Addis Ababa University in Partial Fulfilment of the Requirements for the Degree of Masters of Science in Civil Engineering (Structures Major)

May, 2016

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DECLARATION

I declare that this thesis entitled ―Provision of underpass pedestrian cross in Addis Ababa Railway Construction Project is the result of my own original work except as in references. The thesis has not been accepted for any degree and is not concurrently submitted in candidature of any other degree in any other university that I know of.

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ACKNOWLEDGMENTS

Firstly and foremost, I would like to express my sincere gratitude and appreciation to my advisor, Dr. Asnake Adamu, for his close support and valuable comments during my thesis work without whom it would have been difficult to complete. I really value his contribution for the successful completion of this thesis.

I am also greatly indebted to acknowledge Dr. Esayas G/youhannes for availing himself whenever I needed his support.

I thank the Ethiopian Road Authority (ERA) for giving me a rare opportunity to study for this Master's Degree course under the government sponsored program.

I would like to thank all the provided the requested information including professional guidance. Those who provided valuable support included my, Senior Engineers from Ethiopian Road Authority (ERA), Ethiopian Railway Corporation, Addis Ababa City Roads Authority (AACRA), Net Consulting Engineers and Architects and some members of staff of Addis Ababa Institute of Technology and the general public.

I also would like to thank D/Commander Shambel Mulusew and A/Sagien Hussien Mohammed, the members of Addis Ababa Police Commission, Bole District Community Policing Department who availed traffic accidents data.

Finally, I like to appreciate my families for their encouragement and help.

ABSTRACT

The Addis Ababa Light Rail Transit (AA-LRT) Project consists of E-W & N-S lines, with the total length of the mainline 31.025km has been operational since the end of 2015. The project is implemented to solve the transportation problem of the city at large, but it is identified that the drawback regarding the pedestrian cross is the major cause of the traffic accidents.

This problem of pedestrian cross needs a critical solution for the sustainability of the railway service and integrated traffic flows. The provision of underpass pedestrian crossing enhances safety of pedestrians, save time and solve traffic conflicts. An oral interview questionnaire was administered by interviewing pedestrians who were crossing on the railway station as part of demographic survey. The oral interview also included the vendors selling their goods on the underpass pedestrian crossing and business people with hawkers surrounding the underpass pedestrian crossing. As alternative recommendation, to solve the above problems providing an appropriate path for pedestrians is vital. Constructing pedestrian underpass crossing on the Addis Ababa Railway Construction Projects is one solution for preventing the railway traffic accident in Addis Ababa.

This solution or facilities can represent one of the most important elements of a community's non -motorized transportation network. These structures are built in response to user demand for safe crossings where they previously did not exist. They are provided at locations where there are hazardous pedestrian crossing conditions like in areas where there are few or no gaps in the traffic stream, conflicts between motorists and pedestrians and in locations where large numbers of school children cross busy streets.

The final sample design of Reinforced concrete Bridge superstructures having clear span length of 10 meters and 4.0 meters high bridge clearance with stair and ramps. The pedestrian underpass crossing is located at Ayat – Megenegna and Kality – Meskel Square at every 300m provide pedestrian crossing. Safe and Convenient access to business, civic activities and other modes of transportation.

NOTATIONS

A_s	Area of steel
A_c	Area of concrete
A_g	Gross area
a	Depth of equivalent rectangular stress block (mm); the anchor plate width (mm); the lateral dimension of the anchorage device measured parallel to the larger dimension of the cross-section (mm)
b	Width of the compression face of the member (mm); the lateral dimension of the anchorage device measured parallel to the smaller dimension of the cross-section (mm)
d	Distance from compression face to centroid of tension reinforcement (mm)
d_e	Effective concrete slab depth
E	Young's Modulus in (MPa)
E_c	Modulus of elasticity of concrete (MPa)
E_s	Modulus of elasticity of reinforcing bars (MPa)
EI	Flexural stiffness (N-mm^2)
f_c'	Specified compressive strength of concrete for use in design (MPa)
f_y	Specified minimum yield strength of reinforcing bars (MPa)
I_{cr}	Moment of inertia of the cracked section, transformed to concrete (m^4)
I_{eff}	Effective moment of inertia (mm^4)
I_g	Moment of inertia of the gross concrete section about the centroidal axis, neglecting the Reinforcement (mm^4)
L	Span length

M	Bending moment
M_{cr}	Cracking moment (N-mm)
M_u	Factored moment at the section(strength limit state)
M_s	For service limit state
M_{tr}	Maximum moment due to Truck
M_{ta}	Maximum moment due to Tandem
M_{lane}	Maximum moment due to Lane
NL	Number of lane
Q	Statical moment about neutral axis
t_s	Slab thickness
V_{tr}	Maximum shear force due to Truck
V_{ta}	Maximum shear force due to Tandem
V_{lane}	Maximum shear force due to Lane
W	Uniform dead load
W_{DC}	Total dead load on the bridge
γ_s	Partial safety factor of steel
γ_c	Partial safety factor of concrete
Δ	Deflection

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1. INTRODUCTION

1.1 Background

Addis Ababa, the administrative and financial capital of Ethiopia, is experiencing continued growth and change. Economically, the City is transforming from a predominantly administrative and service center into an industrial and financial center. The population of Addis Ababa, according to the 2007 National Census was about 2.74 million people. This is expected to grow to between 4 and 5 million by 2020 depending on the pace of development expected to take place. By this time, the City would have to be capable of handling an urban travel demand of over 6 million trips per day [12].

The constructions of new light railway in Addis Ababa is currently completed in major city areas from North to South and East to West. Since pedestrians are part of every roadway environment, their interaction with traffic should be a major consideration in railway planning and design. However one can easily witness that this major item is seemingly ignored under this huge industry. Indeed the pedestrian crossing doesn't get much attention by parties involved in the construction. Although the Ethiopian Roads Authority (ERA), Ethiopian Railway Corporation and Addis Ababa City Roads Authority (AACRA) which are main governing parties in railway/road construction in the country and in the City, respectively, in many ongoing railway construction projects in our capital and noticeable traffic problems on pedestrian crossing are observed every day. Whenever there is heavy traffic flow in various directions, multi-level intersections are preferable in view of road safety and saving of time. For instance, a road tunnel for the underpassing fast traffic. Some of these requirements and implications are functional. People should be able to cross the road or railway safely and comfortably. Other requirements are psychological.

The most successful shopping sections are those that provide the most comfort, allow uninterrupted flow of pedestrians movement separate from train and vehicle traffic and pleasure for pedestrians. Underpass pedestrian cross systems are systematic underground pedestrian spaces that have multiple functions for transport, public and commercial usage - such as underground shopping streets, and subway stations with underground concourses. If underpasses are used, it has to be well lighted, cleaned, and the area has to be safe from thieves.

The problems of Pedestrian crossing is a basic element in Addis Ababa railway constructions, while it has given less attention. The result of such problems is causing traffic accident, crowded traffic flow and waste of time. The less attention given to Pedestrian crossing is affecting not only pedestrian, but also vehicular/train/ traffics. One can easily imagine how the problem is more dangerous when it happens on the elderly and the disabled ones.

Because of the absence of safe pathways for pedestrians in the City, they are forced to cross streets through inappropriate way risking their life to traffic accident. It is the responsibility of the professionals engaged in such project to look for a solution to alleviate such big problem of the City. The sum of all mentioned practices leads to conduct this research in order to identify the current situation on the problems of Pedestrian crossing at Addis Ababa railway projects.

Figure -1.1: Problems of pedestrian crossing on Addis Ababa railway lines.

In general, this research studied the current situation of pedestrian cross construction in Addis Ababa railway projects. Samples are taken from ongoing railway constructions and a detailed survey conducted on them. Moreover the study reviewed the availability of pedestrian crossing in the current projects with regard to prevailing signed contract documents and work orders. A check made whether these basic parameters were taken into account during the design and construction of the underpass pedestrian cross as part of justification as well.

1.2. Objective and Scope of the study

General Objective:

- The objective of the research is to understand the current situation and visualize the improvement of the situation at hand, to provide feasible solution for pedestrian crossing on Addis Ababa railway projects.

Specific Objectives:

- It is desirable to reach the underpass pedestrian crossing in a safe way.
- To make a design to separate vehicle and pedestrian crossing avoiding traffic accident and can easily cross at any time of the day or night.
- To create extra space for constructing in the pedestrian underpass for different business activities.

Scope of the study:

The Addis Ababa Light Rail Transit (AA-LRT) Project area consists of E-W and N-S lines, with the total length of mainline 31.025km has been operational since end of 2015. It is solving the transportation problem of the city at large. However, the problem of integrated traffic (trains, vehicles and pedestrians) flow has been observed in its operation mainly on lines from Kality to Meskel square and from Hayat to Megenagna.

This study, based on data analysis and evaluating the social aspect of AA-LRT, attempts to recommend the possible design its functionality for the pedestrian crossing to implement the integrated traffic flows.

1.3 Contents of the Thesis

The thesis is organized in details as follows:

• Chapter one deals with the introductory section of the study where the background, objective and contents of the study are presented.

• Chapter two briefs literature review of the Choice of Underpass Pedestrian crossing; Choice of locations, design considerations and Construction details.

• Chapter three discusses the assessment of the Addis Ababa Light Rail Transit Practice and observed problems

• Chapter four Solution to Addis Ababa Light Rail Transit design of the conventional reinforced concrete bridges and construction practice.

• Chapter five presents the discussion, conclusions drawn from the study and forwards recommendations.

2. LITERATURE REVIEW

2.1 General

Problems of Pedestrian crossing are one of the basic elements in the technical specifications of Ethiopian Railway Corporation, Ethiopian Roads Authority (ERA) and Addis Ababa City Roads Authority (AACRA).

Addis Ababa Light Rail Transit (AA-LRT) Project consists of E-W & N-S lines, with the total length of the mainline 31.025km. These two lines operate on the same rail in downtown area for a total length of 2.662km. The ground line mode is mostly applied in rail laying, while elevated line and underground line are also applied in some sections. The entire line has 39 stations, consisting of 9 elevated stations (including 5 in the section that uses the same rail, 1 on E-W line, and 3 on S-N line), 2 underground stations, 1 semi-underground, and the rest are all ground stations [12].

Entirely open, except for where pedestrians will pass directly under the tracks, a more central location for residents, students and the local shopping precinct. The safest possible means of separating pedestrians from trains, this option is strongly supported by train drivers, disability experts, safety and risk experts, designers and traffic police.

Direct crossing of the railway underpass provided Stair, ramp access and no waiting time – can cross at any time of the day or night. To provide protection light and security cameras are incorporated in the proposed design. The height of both security cameras and lights are over and above the standard, and lighting is recessed into the wall. Fencing to prevent pedestrian access into the rail corridor.

In many cases slow traffic has to cross roads for fast traffic. If there is heavy traffic flow in various directions, multi-level intersections are preferable in view of road safety and saving of time. Therefore we have tried to develop for design for user requirements and implications for designing underpasses. Some of these requirements and implications are functional.

2.2 Choice of Underpass Pedestrian crossing

2.2.1 Choice of Location

The choice a location of underpass pedestrian crossing shall be supported by analyses of alternatives with consideration given to economic, engineering, social, and environmental concerns as well as costs of operation related such as maintenance and inspection.

Attention, commensurate with the risk involved, shall be directed toward providing for favorable underpass pedestrian crossing locations includes:

- ✓ Facilitate practical cost effective design, construction, operation, inspection and maintenance.
- ✓ Provide for the desires level of traffic service and safety.
- ✓ Permits as perpendicular a crossing as possible.
- ✓ Has a good foundation condition.
- ✓ Minimize adverse railway impacts.

2.2.2 Land use along Project Area

The rate of trip making within the area of study depends primarily on land use which in conjunction with socioeconomic characteristics of the area together serves to heighten travel demand. Three characteristics of land use that have been found closely related trip generation are density, character and location of land use activities. Trips related to education, business and work have been highly responsible for flows on the underpass pedestrian crossing. Land use along this section can be classified as [18]:

i. Commercial or office business, banks, fuel station, supermarkets, grocery stores, butcheries, pharmacists and etc.
ii. Residential: The road connects a number of middle class estates and informal settlements lying along the periphery of these middle class income estates. The area has residential building consisting single structure and some multi-storey buildings.
iii. Education: There are various educational institutions.
iv. Recreational: The area has both local bars and big clubs which offer entertainment to the residents of Addis Ababa City.
v. Institutional: The area has both governmental and private owned institutions.

Different land use produces different trip rates. For example a residential area with high density dwellings can produce more trips than the one with a low density of dwellings. On the other hand, low density areas may represent dwellings of the affluent society which may produce a larger number of private trips [18].

2.2.3 Investigation for Underpass Pedestrian crossing

The aim of the investigation is to select a suitable site from possible alternatives at which a bridge can be built economically, at the same time satisfying the demands of safety, traffic, the stream, if any and aesthetics. The investigation for a major bridge project should cover studies on technical feasibility and economic consideration and should result in an investigation report. The success of the final design will depend on the thoroughness of the information furnished by the officer in charge of the investigation.

2.2.4 Factors affecting of the provision of underpass pedestrian crossing and Choice of cross-section

There are a large number of factors affecting the choice whether to provide a subway, and if so the type of cross-section. For this reason it is preferable that each case is considered on its merits having regard to the particular local situation. The following factors have been found to be significant in such consideration process:

- ✓ Volume of pedestrian traffic
- ✓ Whether the access route is to a school, playground or other local amenity
- ✓ Speed of vehicles and trains on the road and the volume of traffic including the proportion of heavy goods vehicles and trains
- ✓ Location, convenience and safety of alternative routes for pedestrians
- ✓ Use by children, elderly people, visually impaired people and disabled people including wheelchair users, and people with prams and pushchairs
- ✓ Environmental aspects
- ✓ Other aspects particularly relevant to the local situation
- ✓ Effects of changes in local land use over the next 15 years including any prospective recreational routes for pedestrians

The line of the underpass and its accesses should preferably be close to the main line of travel for the majority of underpass pedestrian cross users in order to maximize the use of the facility. The underpass should be kept as short as possible. Where the number of pedestrians is very large an option might be to raise the level of the road to reduce the height and length of pedestrian access stairs and ramps.

Personal Security Aspects

Wide approaches, underpass pedestrian cross alignments with good through visibility, and good lighting, all within the view of passing pedestrians and passing traffic, will help to minimize pedestrians' fears for their personal safety. Underpass and their accesses should be designed to avoid places of concealment in the interests of personal security [16].

Attractiveness and good design are important factors in developing the use of underpass pedestrian crossing. It has been found that frequent cleaning and maintenance to preserve appearance are vital in this respect, particularly in the early life of the subway. Finishes should be of high standards, good in appearance and easy to maintain throughout the life of the subway. Physical barriers may be necessary in some locations to prevent cars and motorcycles being driven into subways or subway approaches [16].

2.2.5 Cross-section of Underpass Pedestrian crossing

The pedestrian subway cross-section may be used:

A wide section: suitable for those situations where a subway forms an extension to footpath system 10.0m in width carrying large numbers of pedestrians or where for aesthetic reasons the normal section is not considered to be suitable:

- If circular or other shaped sections are proposed, they should circumscribe the rectangular sections with dimensions.

- Sight distance of 4.0m or more should be provided at corners and changes of direction. For calculation purposes, pedestrians can be assumed to be 0.4m away from an adjacent vertical wall. Inside corners rounded off to a radius of 4.6m will meet these criteria shows on figure 2.1.

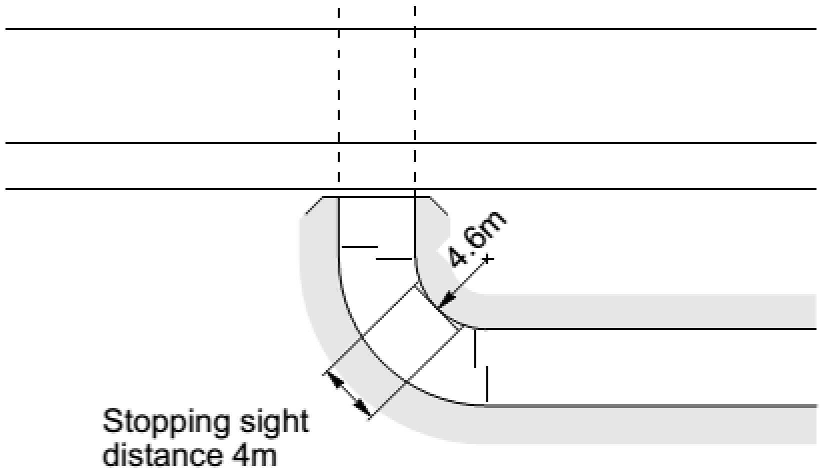


Figure -2.1: Stopping sight distances for pedestrians [12]

2.2.6 Access to the underpass pedestrian crossing

2.2.6.1 General

Pedestrians should be able to enter and leave the subway very easily. Thereby one should not only take account of the ‘’ordinary’’ user but also of handicapped people and parents with prams. Easy accessibility is not only important for comfortable use. It also contributes to feelings of security. Physical obstacles such as staircases hamper the access of the subway for cyclists and police cars so that regular police patrols are impeded. For this reason ramps are to be preferred. Also ramps have the advantage that the total time passed in the underpass is shortened. Likewise the feeling of spaciousness increases and consequently the underpass is perceived.

Access to the underpass pedestrian crossing may be via ramps or stairs which may be straight or helical. Consideration should be given to providing both ramps and stairs to suit able-bodied, people with prams and pushchairs, those with heavy shopping or luggage, visually impaired people and disabled people including wheelchair users.

Access ramps or stairs should normally be the same width as the underpass(subway). The thresholds of all subway accesses, tops and bottoms of flights of stairs, should be provided with a system of tactile paving to assist visually impaired people.

2.2.6.2 Access Ramps

Ramps should not be allowed to run into the subway beyond the threshold. Landings should be provided at changes of direction, and changes of gradient. Landings should be used, even on straight ramps, so that the total rise between landings is not greater than 3.5m. Landings should normally be the same width as the ramp, and 2.0m or more long measured along the center line of the landing. All landings should be approximately horizontal, and adequately drained. Gradients of 5% or shallower are preferred for access ramps where significant numbers of disabled persons or heavily laden shoppers are expected to use the subway. In other situations gradients shallower than 8% are preferred, but gradients up to 10% are permitted for short lengths in exceptionally difficult sites. Stepped ramps may also be considered at exceptionally difficult sites although wheelchair users find stepped ramps difficult to negotiate [16].

2.2.6.3 Access Stairs

The headroom between any ceiling and stair measured vertically should not be less than the height of the subway [16].

Stair flights should normally comprise no more than 20 steps between landings. The landings should normally be the same width as the stair, and preferably 1.8m deep, or a minimum of 1.2m depth in restricted sites. There should not be more than 3 successive flights without a change of direction of 30 degrees or more at a landing. All landings in this case should also be approximately horizontal, and adequately drained. Stair flights limited to 9 steps are preferred where significant numbers of disabled persons are expected to use the stairs. Stair pitch should be uniform for a subway system, with steps of equal rise [16].

Nosing on the stairs should be rounded to a 6mm radius without overhang, and should be colour contrasted from the rest of the step. The stair elements; riser, going, nosing and pitch are illustrated in Figure 2.2.

Table -2.1: Dimensions for straight stairs

Riser (mm)			**Going (mm)**			**Pitch (degrees)**	
Min	Max	Optimum	Min	Max	Optimum	Max	Optimum
100	150	130	280	350	300	33	27

Source: [16]

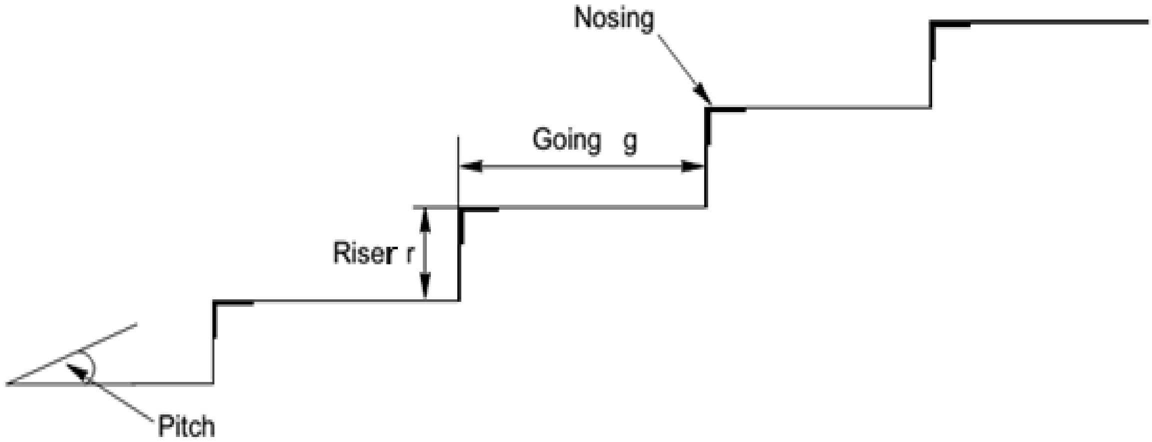


Figure -2.2: Stair elements

2.2.6.4 Visibility

With regard to the access to the subway, the importance of visibility as a means to increase the feeling of personal security is also relevant. This implies a clear view of the immediate environment of the subway and the possibility of being seen by others. Besides the advantage of a better visibility this also resulted in the application of short and rather steep ramps [16]. Visibility is also stimulated by appropriate design. Continuity of interior and exterior space can be obtained by the application of ramps and stair case by gradual change from daylight to artificial lighting or by gradually adjusting the pavement to the situation outside (See Figure 2.3 and 2.4).

Figure -2.3: Visibility for underpass pedestrian crossing that connects the two sides of walkway in Addis Ababa railway [9].

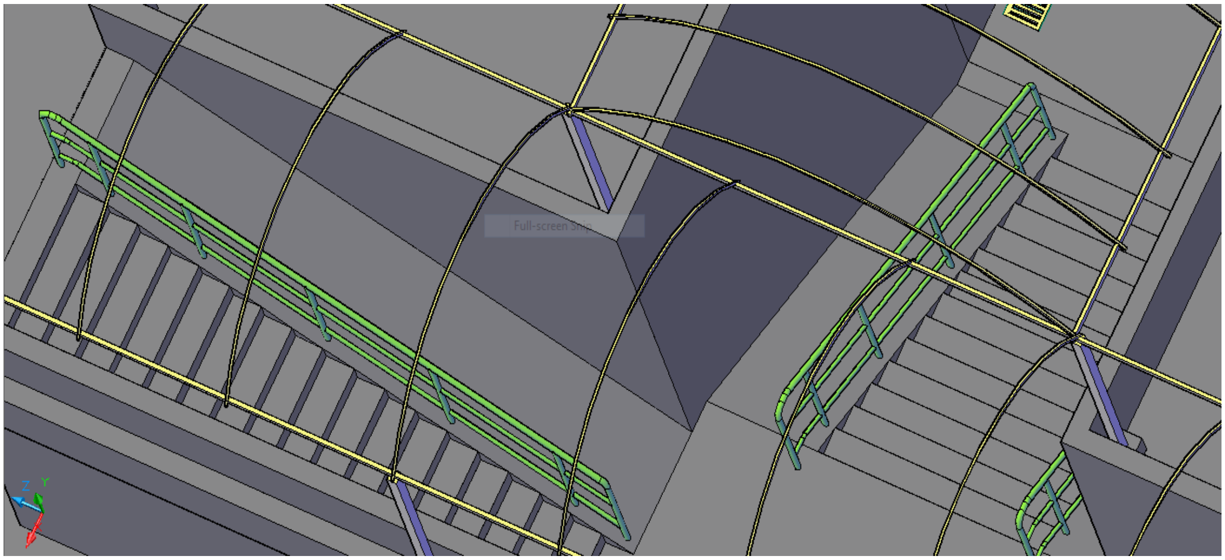


Figure -2.4: Visibility for underpass pedestrian crossing that connects the two sides of walkway in Addis Ababa railway.

2.2.7 Principles of Pedestrian Flow

The qualitative measurement of pedestrian flow is the same as the one used for train(vehicular) flow, such as the freedom to choose desired speeds and to bypass others Highway Capacity Manual(HCM, 2000). Other measures that are related to the pedestrian flow include the ability to cross a pedestrian traffic stream, to walk in reverse direction of a major pedestrian flow, to maneuver generally without conflict and changes in walking speed. Other factors that contribute to perceived level of service are comfort, convenience, safety, security and economy of walking system [18].

2.2.8 Pedestrian Speed-Density Relationship

According to Highway Capacity Manual (HCM, 2000), the relationship between speed, density and volume is similar to that of train(vehicular) flow. As the volume and density increase, pedestrian speed declines. As density increases and pedestrian space reduces, the degree of mobility afforded to each individual declines, as well as the average speed of pedestrian (See Figure 2.5) [18].

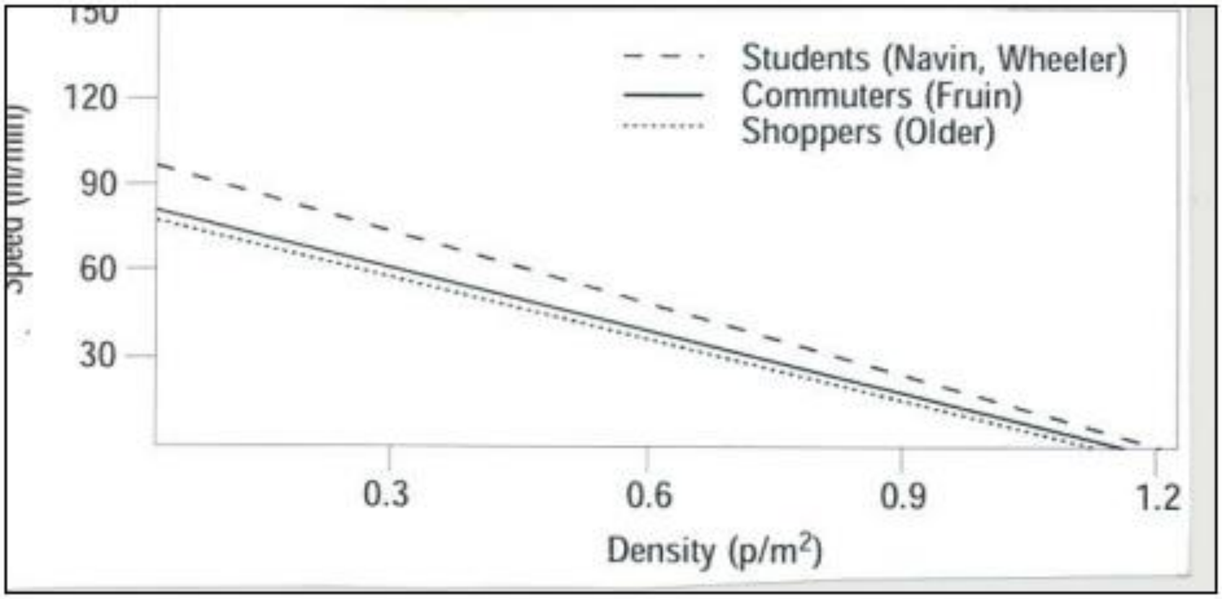


Figure -2.5: Pedestrian Speed-Density Relationship [18].

2.2.9 Pedestrian Speed-Flow Relationship

The relationship between pedestrian speed and flow (see Figure -2.6). The curves shown are similar to the vehicle flow curves. It means that when there are few pedestrians on a walkway or when pedestrians are at low flow levels, there is space available to choose higher walking speeds. As the flow increases, speed declines because of closer interactions among pedestrians [18]. When critical level of crowding occurs, movement becomes difficult and both flow and speed decline Highway Capacity Manual (HCM, 2000).

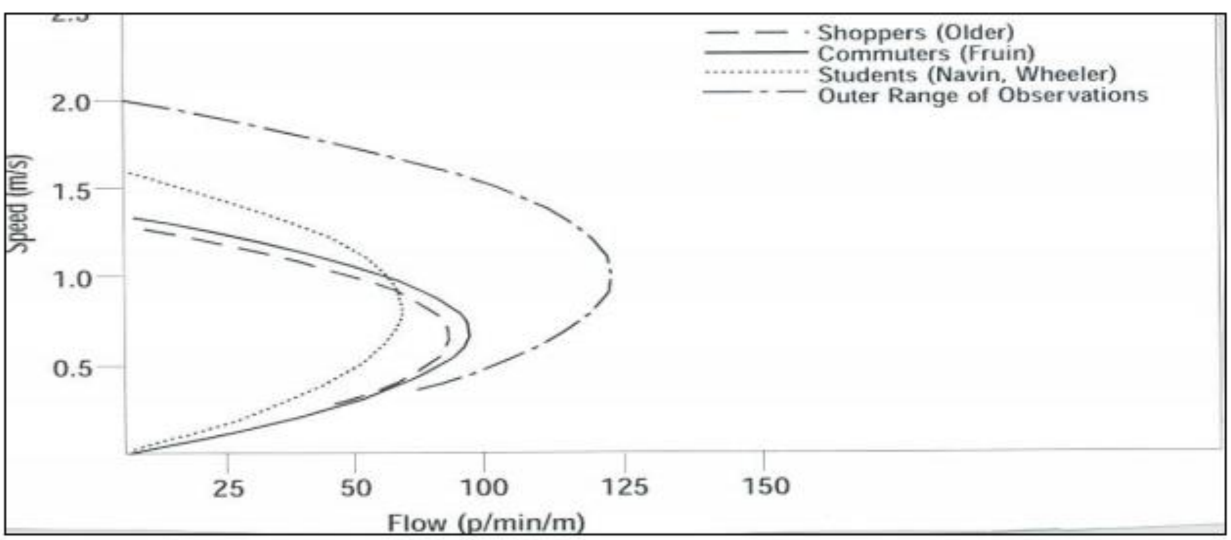


Figure -2.6: Relationship between Pedestrian Speed and Flow [18]

2.2.10 Pedestrian Speed- Space Relationship

Figure -2.7 shows the relationship of walking speed and available space and suggests some point of boundaries. The outer range of observation shown in figure indicates that an average space of 1.5m²/p, even the slowest pedestrians cannot achieve their desired walking speeds. Faster pedestrians, who walk at speed of up to 1.8m /s, are not able to achieve that unless average is 4.0m²/p [18].

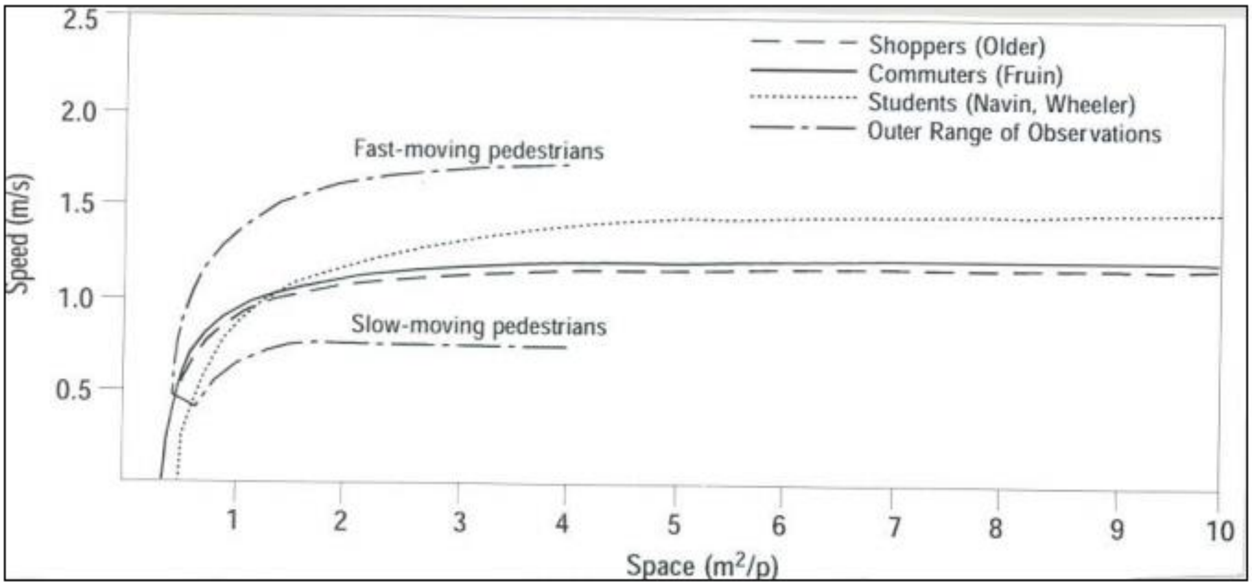


Figure -2.7: Pedestrian Speed- Space Relationship [18]

2.2.11 The Safe System approach and pedestrian safety

Conventional analysis of road traffic injury risk has considered road users, vehicles and the road environment separately. There is also a tendency among researchers and practitioners to focus on one or few factors, when in reality several interacting factors typically define any specific road traffic context. This uneven focus can limit the effectiveness of road traffic injury prevention efforts and may lead to an emphasis on interventions that leave pedestrians at risk. The Safe System approach (see Figure -2.8) addresses risk factors and interventions related to road users, vehicles and the road environment in an integrated manner, allowing for more effective prevention measures. This approach has been shown to be appropriate and effective in several settings around the world, in some cases facilitating road safety gains where further progress had proved to be a challenge [18].

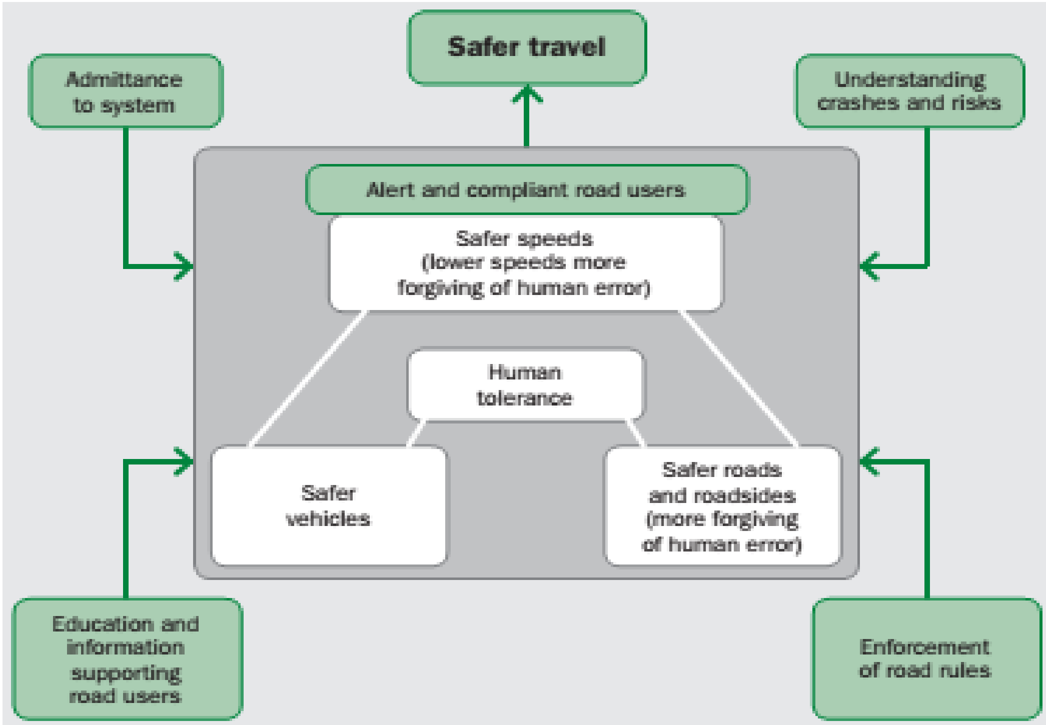


Figure -2.8: Safe system approach [18]

2.2.12 Advantages and Disadvantages of underpass pedestrian cross:

Advantages of underpass:

- Safest option for stations with poor sight distances, a high number of children and the elderly.
- Pedestrian and train(vehicular) traffic are channelized separately allowing traffic to move freely, with less interruption.
- Because conflicts between pedestrians and train(vehicles) are eliminated, the potential for accidents is reduced.
- A more direct (shortest) journey between destinations.
- Safe and convenient access for people with disabilities, parents with prams.
- The underpass is user friendly for the disabled who include those who use the wheelchairs.
- Direct crossing of the railway underpass provided Stair, ramp access and no waiting time – can cross at any time to provide protection light and security cameras of the day or night.

Disadvantages of underpass:

- ✓ Very costly – involves bridge construction
- ✓ Potential safety concerns to users
- ✓ Drainage handling issues and possible need for pump system
- ✓ Potential “hang-out” area

2.3 Design Consideration

2.3.1 Design and Construction Techniques:

An attractive and easily accessible to underpass pedestrian cross makes high demands upon the dimensions of the alignment. The underpass will be a straight from one side to the other allowing direct views and open feeling. Also adequate illumination is important in order to attain road safety and to create a sufficient degree of spaciousness to prevent feelings. Further attention ought to be paid to good upkeep and sufficient ventilation. All these aspects contribute to a positive perception of the underpass as a whole.

The structural shape and cross section of the lining are determined by geological conditions and the direction of the primary loads on the lining. Underpass pedestrian crossing lines are designed for the least favourable but realistic combinations of primary, secondary, and special loads and their effects on the construction materials. Primary loads, such as rock pressure, act on the lining constantly or regularly. Secondary loads act for short times or periodically, and special loads are mainly seismic in nature. The underpass will be wide and open, with 10.0 meters wide paths stair and ramps.

The design is entirely open, with the only underground walkway. It features gentle slopes and wide 10.0 meter paths, with a 4.0 meter high bridge clearance.

- ✓ Installation of emergency (hands free) phones.
- ✓ Convex mirrors on bends to see around corners.
- ✓ Quality lighting, recessed into the wall to prevent vandalism.
- ✓ Straight ramps with no hidden corners.
- ✓ Easy to maintain, durable and vandal resistant materials will be used and will be finished in light colours to reflect and refract the light.
- ✓ Designed with sloping banks to approach paths instead of retaining walls to maintain sight distances and create open entrances.

The design service life of the main structure works shall be 100 years. Safety class of the main structure members of the station is Class-I. The structure design should guarantee sufficient strength, rigidity, stability, and durability during construction and service period.

2.3.2 Design Objectives

The objective in a bridge design is safety, serviceability, economy, constructability and aesthetics.

a) Safety: - the primary responsibility of the engineer is to ensure public safety in the design by ensuring adequate structural [12].

b) Serviceability: - consists of satisfying requirement of deformation, durability, inspect ability and maintainability [12].

Deformation: - Bridges should be designed to avoid excessive deformation that causes undesirable structural or psychological effects. Limits on deflection or minimum depth to consider are given in codes (AASHTO 98, Articles 2.5.2.6.2 and 2.5.2.6.3 respectively) [12].

Durability: - contract document specify quality of materials to be used and standards of fabrication and erection of elements to ensure durability. Self-protecting measures of the structure from the effects of the weather will be taken during design and construction [12].

Inspectability: - inspection ladders, walkways, catwalks and covered access holes will be provided where other means of inspection are not practical [12].

Maintainability: - structural system whose maintenance is expected to be difficult should be avoided [12].

c) Economy: - structural types span length and material should be selected based on cost. The cost of future expenditure during the projected service life of the bridge should be considered [12].

d) Constructability: - bridges should be designed in a manner such that fabrication and erection can be performed without undue difficulty or distress and that construction force effects are within tolerable limits [12].

e) Aesthetics: - bridges should complement their surroundings, be graceful in form and present an appearance of adequate strength. Because the major structural component are the largest parts and are seen first, they determine the appearance of a bridge. Although beauty in anything is somewhat subjective and indefinable, experience has shown that there are some general guidelines which, if followed, will result in a structure of improved appearance. Bearing in mind that a bridge may well serve for 100 years or more the designer should become familiar with the aesthetic techniques that may be used to improve the appearance of a bridge so that the bridge will be a credit to its designer and to itself for many decades to come. The following are some special areas of concern [12].

The different limit states may be defined now:

Strength limit state: - is meant to ensure that strength and stability, both local and global, arc provided to resist the specified statistically significant load combination that a bridge is expected to be subjected to in its design life [12].

Service limit state: - is a restriction on stress, deformation and crack width under regular service conditions [12].

Fatigue and fracture limit state: - the fatigue limit state is restriction on stress range as a result of a single design truck occurring at the number of expected stress range cycles. The fracture limit state is taken as a set of material toughness requirements [12].

Extreme event limit states: - this is taken to ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vehicle, vessel, possibly under scoured conditions [12].

Ductility: - the structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states prior to failure. Ductile structures give ample warning before they lose load carrying capacity whereas brittle structures collapse without giving warning [12].

Redundancy: - multiple load path and continuous structures should be used unless there are compelling reasons not to use them. Indeterminate structures survive overloads and extreme events due to multiple load paths and redistribution of internal forces [12].

2.3.3 Expansion Joints in Bridge concrete structures

Joints are necessary in concrete structures for a variety of reasons. Not all concrete in a given structure can be placed continuously, so there are construction joints that allow for work to be resumed after a period of time. Since concrete undergoes volume changes, principally related to shrinkage and temperature changes, it can be desirable to provide joints and thus relieve tensile or compressive stresses that would be induced in the structure. Alternately, the effect of volume changes can be considered just as other load effects are considered in building design. Various concrete structural elements are supported differently and independently, yet meet and match for functional and architectural reasons. In this case, compatibility of deformation is important, and joints may be required to isolate various members.

2.3.4 Main design principles and standards

1) The structure design should satisfy the principles of safety, serviceability, durability and economic efficiency. All main structural members should meet the requirements of both ultimate limit state and serviceability limit state. Structural design should meet the serviceability requirements of clearance, construction technology and operation and at the same time guarantee the durability of structure [12].

2) According to the engineering geology and hydrogeologic condition of the Project and requirements of urban comprehensive planning, in combination with the situation of existing buildings, environmental conditions, pipelines and road traffic, and by comprehensive comparison of technology, economy and service function, reasonable construction methods and structure forms should be adopted for structural design [12].

3) Structural design should meet the requirements for checking calculation of strength, rigidity, stability and anti-floating and crevice developing, and satisfy the requirements of construction technology [12].

4) Calculation of structural strength, rigidity and stability of underground structure should be conducted for construction and normal service stages. Checking calculation of crevice width should be conducted for concrete structure. When the impact of seismic load or other accidental load is included, checking calculation of crevice width is not needed. Allowable value of maximum crevice width is combined according to load effect standard and is under control inconsideration of long-term impact influence, allowable value of maximum crevice (narrow crack) width: Checking calculation of structural crevice width of downstream face is no more than 0.3mm. Checking calculation of structural crevice width of upstream face is no more than 0.2mm, internal element no more than 0.3mm [12].

5) When significant change occurs to structure, groundwork, foundation or load of underground section tunnel, or when deformation joint must be set according to seismic requirements, reliable engineering measures should be adopted to ensure that no differential settlement of normal train operation or curvature change of rails will be caused by the structure of the two sides of deformation joint [12].

6) When construction of underground station has impacts upon buildings (structures), settlement should be estimated in advance and reliable technical schemes and construction methods that can ensure the normal service of buildings (structures) should be adopted. The calculation of allowable settlement and secondary stress to buildings (structures) should be checked based on types and foundations of different buildings and according to relevant codes, specifications and requirements [12].

7) The major structure is reinforced concrete structure. Concrete grade is C-30. Waterproof concrete is adopted for roof, base slab and sidewall. Materials of Class I fire resistance rating are adopted for major force support element [12].

8) Effective measures against stray current should be taken for underground station structure to prevent the corrosion to structures by stray current [12].

9) Structural waterproof design should be based on engineering geology, hydrogeology, seismic intensity, environmental condition, structure form, construction technology and source of materials and should follow the principle of "waterproofing, drainage, interception, and blockage combination, alternation between rigidness and flexibility, adaptation to the local conditions, and comprehensive treatment" [12].

2.3.5 Rail Transit Load

Where a bridge also carries rail-transit vehicles, the owner of the railway shall specify the transit load characteristics and the expected interaction between transit and highway traffic.

If rail transit is designed to occupy an exclusive lane, transit loads should be included in the design, but the bridge should not have less strength than if it had been designed as a highway bridge of the same width.

If the rail transit is supposed to mix with regular highway traffic, the Owner should specify or approve an appropriate combination of Railway transit and highway loads for the design. Railway Transit load characteristics may include:

- Loads,
- Load distribution,
- Load frequency,
- Dynamic load allowance, and
- Dimensional requirements.

2.3.5.1 Dynamic Load Allowance

Dynamic effects due to moving vehicles shall be attributed to two sources:

- Hammering effect is the dynamic response of the wheel assembly to riding surface discontinuities, such as deck joints, cracks, potholes, and delamination, and
- Dynamic response of the bridge as a whole to passing vehicles, which shall be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or to resonant excitation as a result of similar frequencies of vibration between bridge and vehicle.

2.3.6 Pedestrian Loads

A pedestrian load of 4.0kN/m² shall be applied to all sidewalks wider than 0.6 m and considered simultaneously with the vehicular design live load. To avoid accidents for bridges wider than 2.4 m, provision shall be making for an additional axle load [18].

Where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance and/or other incidental vehicles, these loads shall be considered in the design. The dynamic load allowance need to be considered for these railways, but not considered for the vehicles [18].

2.3.7 Exterior Design

For the sake of security of slow moving traffic, clarity of arrangement of the exterior area is important. Besides good visibility of the entrance, clarity may also be obtained by the application of the principle of continuity in the design of the pavement. Such effects can also be attained by uniformity in the use of materials. A proper indication of the pathway is also desired for optimal use of an underpass. Another way to increase its use is adequate route indication and the clear marking of the entrance by signs and name-plates [5].

Clarity of arrangement of the underpass environment is important not only for functional reasons, but also from the point of view of social security. From studies of the effect of architectural design on the prevention of crime it is known that there exists a clear relationship between environmental characteristics and feelings of particularly, dark, deserted and derelict areas evoke feelings of unpleasantness. Such areas lack social control, a feeling that there are people who can see what is going on and who, if necessary, can interfere or come to aid. The following factors may contribute to an environment with a strong sense of safety and social control [5].

Integration of the Subway into the Environment as a whole

Underpass must be seen as a structural element of the total spatial concept. It should not be designed as a single object because this may lead to an isolation location with regard to its residential surroundings. This in turn may elicit a feeling of being in no-man's land. From the point of view of safety it is important that the underpass should not be perceived as a ‘’foreign object’’ in a deserted traffic area but that it should be part of the residential area and as such included in the sphere of influence of the inhabitants.

For instance through the proximity of dwellings or other facilities to attract people a visible audible and perceptible control is ensured. In this way a ‘’sense of ownership’’ is generated by which the degree of involvement of the residents increases and feelings of security will be strengthened [5].

Residential Area versus Traffic Areas

The impression of desertedness which is often evoked in the immediate environment of underpasses is also caused by its characteristics of traffic rather than a non-traffic area. Traffic areas can be described as areas primarily designated for traffic-circulation purposes. On the other hand, in traffic-restraint areas residential functions predominate. Here people linger, children are playing and activities take place. The more activities take place, the more people, the greater amount of social control and the less opportunity for criminal activities. To be safe a place should have attractive features to encourage people to make use of it, because the presence of people has itself a positive effect in attracting other people [5].

Distinctness of the Area

Besides the presence of people it is important that the immediate environment of a subway has clearly recognizable functions. Places with an indistinct function and an ambiguous character evoke feelings of unpleasantness and desolation, because nobody seems responsible [5].

The public areas in particular are very vulnerable to vandalism or other misbehaviour since these are neither public nor private. Therefore it is necessary that differences in functions should be clearly defined in traffic and traffic-restraint areas, public and private places. For example, ambiguity can be taken away by establishing clear indications of the planned functions of the surroundings [5].

2.4. Construction Details

2.4.1. Surface Finishes

The clarity of arrangement of the exterior area is important. Besides good visibility of the entrance, clarity may also be obtained by the application of the principle of continuity in the design of the pavement. Such effects can also be attained by uniformity in the use of materials.

a) Walls:

The walls are the most conspicuous and vulnerable areas and their finishes will affect the whole character of the subway. Some finishes are difficult to keep clean and have poor quality of light reflection. Important considerations in the selection and specification of finishes are their resistance to vandalism and the ease with which any graffiti can be removed. For these reasons, porous open surfaced materials such as facing bricks and exposed aggregate finishes are best avoided. Mosaics and other hard impervious surfaced materials have performed well in the past. They are reasonably graffiti-proof and easy to clean [16].

In situ structural concrete and precast concrete are more prone to graffiti; but this can be discouraged by the application of suitable plastic paints to make walls impervious and easier to clean. Bold designs with bright multiple colours in irregular or random patterns, and murals with themes suggested by children can help to create an atmosphere that the subway is well-used and therefore safer. This has also been found to deter vandalism [16].

b) Floors Ramps and Stairs:

Finishes may be subjected to all weather conditions and to salting and gritting in winter. They should have an adequate and durable slip resistance both when wet and when dry. The same advice should be followed for footpaths, access stairs and access ramps [16].

c) Ceilings:

Concrete soffits should be treated to maximize the amount of light reflection. Finishes known to have been successful include plastic paint with a matt white finish, Tyrolean and cement sprays. Suspended ceilings should not be used [16].

2.4.2. Lighting

Lighting affects visibility and clearness. To promote road safety and reduce feelings of anxiety or claustrophobia it is important that people can see what is going on. Better visibility may even reduce the actual crime rate. The amount of light which is appropriate depends on the situation. For example, different lighting levels are suggested for highways and for residential areas. Daylight penetration into the subway entrances should be utilized wherever possible, with surface finishes chosen to enhance daylight illumination [16].

Artificial lighting should always be provided for use in the hours of darkness both inside the subway and on the subway approaches. Continuous use of lighting, in the daytime also, will encourage subway use in many cases. Various unsocial and criminal activities take place in the underpasses at night time although there are lighting facilities. Generator service is also provided for the time of load shedding [16].

Vandal-proof lighting systems should generally be used. Luminaries recessed into the ceiling or into the tops of walls have been successful in the past although surface mounted corner light fittings should be satisfactory in most situations provided they do not unduly encroach into the minimum cross-section required by this standard. Appropriate location of the lighting fixtures is also important to avoid vandalism, for instance by recessing lighting into the roof or walls, out of the vandals but in such a way that they can be replaced easily. This prevents delay in repair which often leads to a dismal appearance of the environment. Light covers of toughened or reinforced types of glass should be used to resist the milder forms of vandalism [16].

The provision of lighting makes it more difficult for persons to be hidden from view at night. Lighting is considered a medium level treatment. Good lighting may significantly reduce the opportunity for perpetrators to go unnoticed at night. Due consideration needs to be given to glare for oncoming traffic and light spillage to surrounding or adjacent buildings [16].

Lighting was considered to be an expensive option, particularly when being retrofitted to structures. Given the uncertain effectiveness of this type of treatment careful consideration to the benefit-cost ratio of the treatment would need to be given prior to its implementation [16].

2.4.3. Video Surveillance

The presence of surveillance cameras provides a deterrent to some potential wrongdoers, and there has been a significant use of this type of treatment throughout public areas. However this level of surveillance and response is not always necessary, practicable or cost effective. Given the long distances involved in some of the bridge crossings, clear identification of the perpetrators may be difficult [22].

Some economies of scale may be able to be obtained where surveillance cameras are used for other purposes such as security at train stations. Risk managers would need to weigh the effectiveness or benefits of this type of treatment against the whole-of-life cost, and consider other treatments that may be just as effective and less costly in the long term [22].

2.4.4. Drainage

2.4.4.1 General

Drainage is another consideration for underpass especially when they are below grade. If an underpass is not properly drained then there is the potential for internal flooding in the summer months if water is permitted to pond. Improper drainage causing water seepage into the backfill is the leading cause of retaining wall problems. Lateral earth pressure design is usually based upon drained soil. Saturated soil can substantially increase pressures.

2.4.4.2 The use of land drainage perforated pipes

Perforated pipes are used in draining ground water and Surface water from areas including roads, land reclamation, retaining walls and land drainage. Where land drainage is required to reduce the ground water level permanently in order to enable development to proceed, expert advice should be obtained.

a) Surface water drainage:

When used to control rainfall draining from impermeable ground such as roads and the camber of the paved or concreted area runs rain water in to the permeable fill above the perforated pipe. In this instance the surrounding filter drain material is of course nature to allow surface water to percolate freely through it. However, grit from road surfaces may block the filter which may require cleaning periodically. As an additional precautionary measure to limit soil migration, a geotextile fabric can be incorporated into the filter drain.

- Drainage aggregate is a free draining soil with natural soil filtering capabilities, or a free draining soil encapsulated in a suitable geotextile, or a combination of free draining soil and perforated pipe all wrapped in a geotextile, placed directly behind the modular concrete units.
- Drainage pipe is a perforated pipe used to carry water, collected at the base of the retaining wall, to outlets in order to prevent pore water pressures from building up behind the wall facing modules.
- Levelling base is a layer of specified soil that is placed below the proposed wall to provide a level working surface.
- Non-woven geotextiles are permeable synthetic fabrics formed from a random arrangement of fibres in a planar structure. They allow the passage of water from one soil medium to another while preventing the migration of fine particles that might clog a drainage medium.

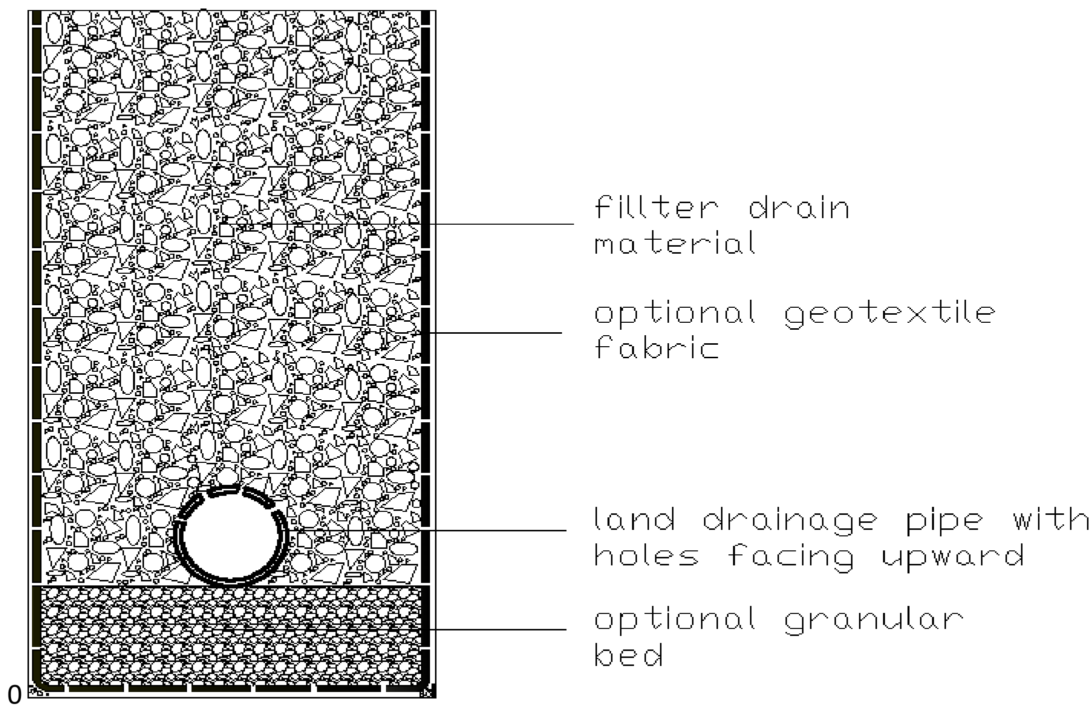


Figure -2.9: Land drainage perforated pipe behind a retaining wall.

b) Dispersal of septic tank effluent:

Perforated pipes may be used to dispose of septic tank effluent by sub-surface irrigation. To allow the dispersal of water of sewage effluent from small sewage treatment works and cesspools into the underground strata, the pipe is laid with the holes downwards. The groundwater table must be below the invert level of the pipe. The permeable fill should be covered with a strip of plastic to prevent the ingress of silt and fine particles.

2.4.4.3 The internal part of drainage system

The floors of pedestrian subways should be cambered with transverse slopes of about 3% and shallow channels on each side. For segregated subways the drainage should be at the edge. It is preferable for the subway to slope longitudinally at a gradient of not less than 0.7% [16].

The drainage system should be large enough to deal with the water and detritus entering the subway from the ramps and stairs. If it is not possible for gravity flow to be used then a pump systems may be required. Though most new pump systems are pretty reliable, it is still a good idea to build in redundancy with a stand-by pump. Lockable or hinged gratings are recommended in situations where vandalism or theft is a problem. Adequate provision should be made for the cleaning and maintenance of drains. They include snow removal, drainage, sweeping and cleaning, and miscellaneous items such as periodic inspections and repairs. In addition, the maintenance cost for the underpass alternatives includes the maintenance of a sump and pumping system. A sump with pump and lift station should only be considered if connecting to an existing storm drain by gravity flow is not practical. The inside structure's floor shall be sloped as required to facilitate draining to the drainage collection system [16].

2.4.5. Handrail

Handrails should be provided on both sides of stairs and ramps. Central handrails may be advisable where the width of stairs or ramps exceeds 3.0m. To assist elderly people and disabled people, the handrail height should be 1.0m above the level surface, 0.9m above a ramp and 0.85m above the nose of a step [16].

People with frail or arthritic hands have difficulty in gripping objects. The most comfortable sections for handrails are round sections between 45mm and 50mm in diameter and there should be a gap of 45mm between the rail and the wall [16].

Where used, bollards and metal railings should be between 1.0m and 1.2m high. The minimum access gaps between these barriers should be 1.2m wide for the passage of wheelchairs and double prams [16]. To assist the visually impaired, the tops of these barriers should be applied with colour contrasting paints (See Figure 2.10).

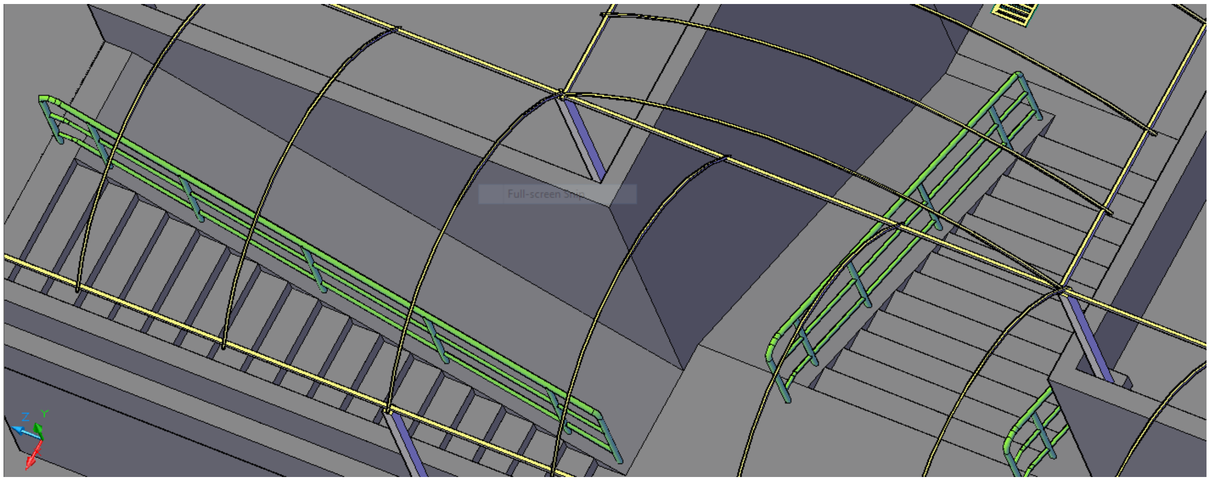


Figure -2.10: Handrail for underpass pedestrian crossing that connects the two sides of walkway on railway.

2.4.6. Markings and Signs

A proper indication of the pathway is also desired for optimal use of an underpass. Another way to increase its use is adequate route indication and the clear marking of the entrance by signs, stairs and ramps by name-plates (See Figure 2.11). The use of this type of treatment is usually accompanied by a broader education and/or enforcement program developed as part of a broad risk management strategy. Signs are considered a low level treatment strategy. The signage should provide a simple message such as “Bridge Under Surveillance” [16].

As an alternative suggestion, signage containing the number of the toll free Crime Stoppers line may prompt other users to advise the police of incidents that may lead to an undesirable event. Signs need to be placed in a conspicuous location to maximise their effectiveness [16].

Figure -2.11: The Signs will indicate directions of underpass pedestrian crossing [9].

2.4.7. Maintenance

The Pedestrian Underpass Structure shall be designed considering ease of maintenance. Lighting fixtures, signage, aesthetic treatments, and other materials proposed for use in the structure shall be included in the design considerations to reduce maintenance frequency and requirements.

For example, lighting fixtures should be recessed with unbreakable lenses or provided with lens protection. Signage should be adequately fastened to supports and out of reach to the extent possible. Aesthetic treatments should be inlaid or anchored and protected by other means from vandalism damage. Concrete should have graffiti protection if warranted, and handrails and exposed metals should be protected from corrosion (galvanized steel is preferred). The Pedestrian underpass structure shall be maintained by and at the expense of the sponsoring Public Agency.

3. ASSESSMENT OF THE ADDIS ABABA LIGHT RAIL TRANSIT PRACTICE AND OBSERVED PROBLEM

3.1. General

3.1.1. Underpass and Overpass Pedestrian Crossing

Pedestrian overpasses and underpasses are bridges and tunnels that allow for uninterrupted flow that is separate from vehicular traffic. This measure is used primarily in areas with high pedestrian volumes.

Several issues arise with implementation of overpasses and underpasses:

- The effectiveness of these approaches depends largely upon the likelihood that they will be used by most of the pedestrians crossing the street. The level of use depends on convenience, security, and walking distances compared with alternative crossing locations. Pedestrians generally do not use these facilities if a more direct route is available. Tall fences and other pedestrian barriers may be used to channel pedestrians to the overpass or underpass. These are not always effective, however, since pedestrians find ways to go around the barriers and cross at intersections.
- Overpasses are suitable when the topography allows for a structure without ramps, for example, an overpass over a below-grade freeway. Overpasses with multiple stairs are not user-friendly for the elderly or disabled pedestrians. Underpasses need to be designed in such a way as to offer a sense of being open and accessible.
- Ramps must be designed to accommodate pedestrians in wheelchairs.
- Underpasses are often dark, secluded places. They may be targeted by gangs or other perpetrators of interpersonal violence, and, for this reason, people who perceive a high risk of assault avoid them. Overpasses and underpasses should be well-lit and secure, to maximize personal security and therefore utilization.

3.2. Underpass Pedestrian crossing

Pedestrian/bicycle underpass serves many users, including bicyclists, walkers, joggers, inline skaters, and pedestrians with strollers, wheelchair users, and others. These facilities can represent one of the most important elements of a community’s non-motorized transportation network. Underpasses provide critical links in the bicycle/pedestrian system by joining areas separated by a variety of “barriers.” Underpasses can address real or perceived safety issues by providing users a formalized means for traversing “problem areas” such a major transportation corridors.

Underground pedestrian systems are systematic underground pedestrian spaces that have multiple functions for transport, public and commercial usage - such as underground shopping streets, and subway stations with underground concourses. Such underground pedestrian systems are normally located in sub centers and regional centers in metropolises and strongly influence city functions. Adopting a systems approach to analyzing urban pedestrian space helps to conceptualize the relationship between underground pedestrian systems and other pedestrian spaces within the broader urban environment.

- Firstly, viewing the city as a complex system, an underground pedestrian system is a vital subsystem of public space systems, expanding space and movement below the surface pedestrian systems.
- Secondly, an underground pedestrian system uses walking as a mode of transport to link and aggregate activities such as retailing and mass transit within an underground setting.
- Thirdly, an underground pedestrian system has an important functional feature through the provision of underground public passageways functioning as a medium that integrates ground level spaces with underground spaces.

Pedestrians prefer to travel on level grade with minor undulations and very few or no major changes in elevation. A below grade crossing is an ideal way to accommodate pedestrian travel especially when the feature to be crossed is an elevated road. This can result in a below grade crossing that is actually at the same grade as the path but under the grade of the intersecting road.

In most cases, these structures are built in response to user demand for safe crossings where they previously did not exist. For instance, an underpass may be appropriate where moderate to high pedestrian/bicycle demand exists to cross a freeway in a specific location. Pedestrian/bicycle are appropriate in railroads areas where frequent or high-speed trains would create at-grade crossing safety issues. They may also be an appropriate response to railroad and other agency policies prohibiting new at-grade railroad crossings, as well as efforts to close existing at-grade crossings for efficiency, safety, and liability reasons.

The fact that this type of structure would convey pedestrian under the traffic on Addis Ababa Railway immediately brings to mind issues like 24-hour a day lighting. Proper lighting is imperative in any underpass built to improve pedestrian security. Whenever possible, any measure to improve illumination in underpass will be implemented for pedestrians to feel safer using them. In the design of the pedestrian underpass crossing that connects the two sides of walkways, besides overhead lighting natural light coming in through skylight in median opening in the ceiling from the outside was also provided (See Figure 3.1).

- Safest option for stations with poor sight distances, a high number of children, the elderly and Alcohol(drugs) impair the behaviour of pedestrians to the extent that they may be a primary cause of accident.
- Pedestrian and train(vehicular) traffic are channelized separately allowing traffic to move freely, with less interruption.
- Because conflicts between pedestrians and train (vehicles) are eliminated, the potential for accidents is reduced.
- A more direct (shortest) journey between destinations.

- Safe and convenient access for people with disabilities, parents with prams.
- The underpass is user friendly for the disabled who include those who use the wheelchairs.
- Direct crossing of the railway underpass provided Stair, ramp access and no waiting time – can cross at any time to provide protection light and security cameras of the day or night.
- Accessibility to business and civic activities and other modes of transportation.

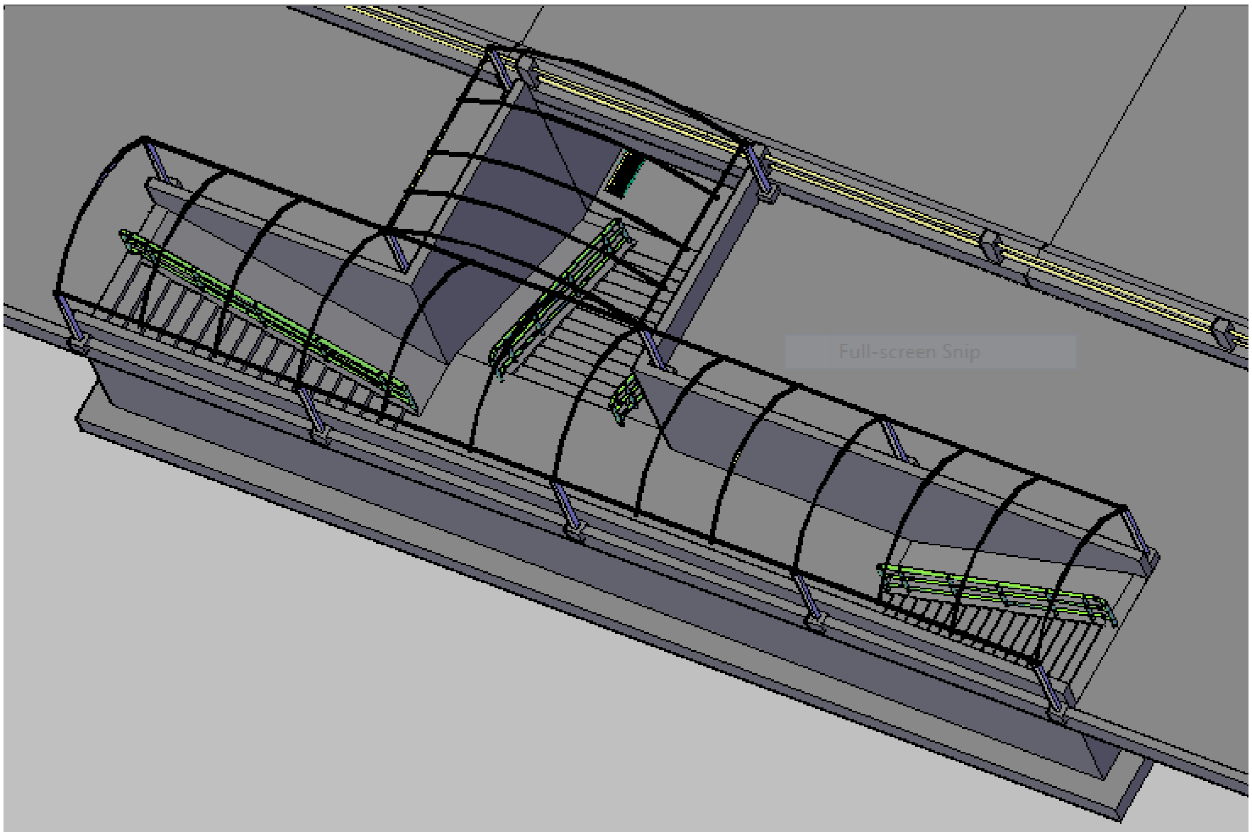


Figure -3.1: Underpass pedestrian crossing that connects the two sides of walkway on railway.

3.2.1 Subways: A solution with problems

In many cases slow traffic has to cross roads for fast traffic. If there is heavy traffic flow in various directions, multi-level intersections are preferable in view of road safety and saving of time. For example a viaduct or road tunnel is very expensive, while a bridge for slow traffic crossing a speedway or railroad requires special effort to reach bridge level, which is rather high to allow sufficient headroom for the under passing fast traffic [5].

Actually, an underpass may be the lesser evil, indeed an underpass for slow traffic is often chosen. Therefore we have tried to develop for designers and planners a checklist of user requirements and implications are functional. People should be able to cross the road or railway safely and comfortably. Other requirements are psychological [5].

Closed spaces may lead to claustrophobic experiences, so measures should be taken to prevent such feelings. Furthermore, precautions are required in order to prevent feelings of insecurity e.g. fear of violence. In the next section of this article we will describe the research material on which this article is based. Subsequently we dwell on the considerations important for the optimal location of the underpass and the design objectives related to the access area [5].

3.3. Overpass Pedestrian crossing on the Railway Stations

Pedestrian overcrossings also respond to user needs where existing at-grade crossing opportunities exist but are undesirable for any number of reasons. In some cases, high vehicle speeds and heavy traffic volumes might warrant a grade-separated crossing. Hazardous pedestrian crossing conditions (e.g. few or no gaps in the traffic stream, conflicts between motorists and pedestrians at intersections, etc.) could also create the need for overcrossings. Overcrossings might also be appropriate in locations where large numbers of school children cross busy streets, or where high volumes of seniors or mobility-impaired users need to cross a major roadway.

Pedestrian overpasses are structures that provide pedestrians, a route over an obstruction such as a road. They allow for the continuous and uninterrupted flow of traffic on both the road and the path. By their nature they are typically supported on columns or other structures that elevate them over the obstacle. Overpasses, particularly those crossing over high-volume roads require large vertical clearance to accommodate the various types of vehicles that may pass underneath. These structures have to be of sufficient width to accommodate two-way traffic which can include walkers, but not motorized vehicles.

An evaluation of the overpass conducted to the following results:

- Just over one third of pedestrians used the overpass. The low usage of the overpass reflected some of the design flaws, as well as the position of the overpass, which raised security concerns among users. Respondents were concerned that the overpass was untidy, poorly and that children loitered on it. Most pedestrians found the overpass to be inconvenient and difficult to access. Consequently, many pedestrians could be seen crossing the road through motorized traffic.
- While the number of pedestrians killed dropped from eight to two after it was constructed, the number of pedestrians seriously injured increased with time. The mixed outcomes associated with this isolated intervention indicate the need for a comprehensive approach to pedestrian safety. Other measures such as reducing and enforcing vehicle speeds, providing raised crossings, providing sidewalks and raising awareness about these measures would have complemented the overpass.

3.3.1 Problems of overpass pedestrian crossing:

- ✓ An elevated, above grade structure is very expensive relative to an at-grade crossing.
- ✓ Ramps connecting to the bridge take up a lot of space.
- ✓ Providing direct access to Taxi and Bus stations could be difficult or impossible and could require obtaining right-of-way.

- ✓ Access to some businesses could be blocked or difficult to reach.
- ✓ Pedestrians may not be willing to walk the extra distance to cross the bridge.
- ✓ Height can be obtrusive into the existing urban landscape, for train electrical system and aesthetically not good (See Figure 3.2).
- ✓ The overpass pedestrian crossing is not user friendly for the disabled who include those who use the clutches, wheelchairs, parents with prams and by cyclists (See Figure 3.3).

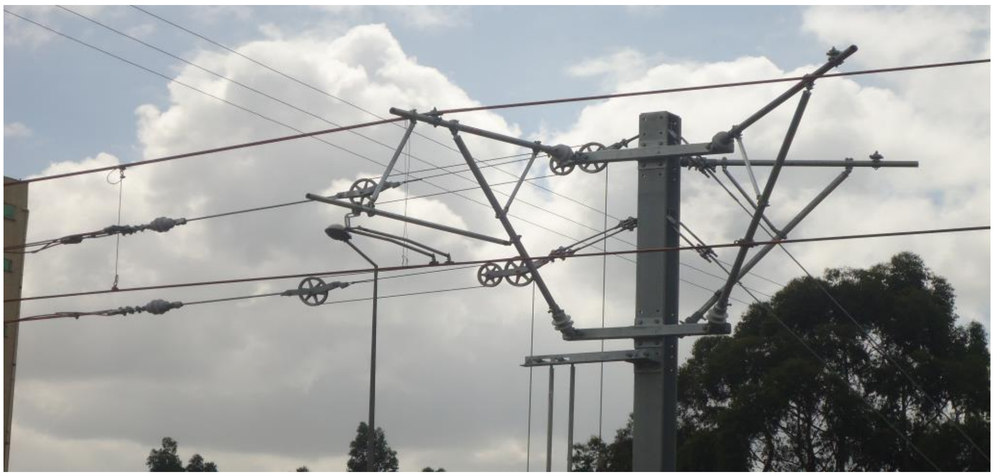


Figure -3.2: Addis Ababa Light Railway Transit Project electrical line.

Figure -3.3: Some Overpass Pedestrians crossing with bicycles [18]

3.4. Survey Interviews

3.4.1 Oral Interviews Questionnaire to Pedestrians

Locations from Kality Station – Gotera Square and Ayat Station - Megenagna Station

An oral interview questionnaire was administered by interviewing pedestrians who were crossing on the railway station as part of demographic survey. The oral interview also included the vendors selling their goods on the underpass pedestrian crossing and business people with hawkers surrounding the underpass pedestrian crossing.

Figure -3.4: Research conduction oral interview on a pedestrians from Kality Station – Gotera Square and Ayat Station - Megenagna Station.

3.4.2 Results on Oral Questionnaire Interviews to Pedestrians

Oral Interviews to pedestrians passing through the pedestrian railway station. The interview was done both during peak hours and off peak hours. During the peak hours, the interview was done from 6.30 - 9.30 am and 4.00-7.00 pm and off peak hour interview was done from 11.00 am -1.00 pm.

3.4.2.1 Pedestrian Profile

3.4.2.1.1 Gender

Out of 154 pedestrians who were interviewed, it was observed that 81 pedestrians were male and the remaining 73 pedestrians were female (See Figure 3.5). This represents almost equal distribution of administering of the oral interview questionnaires.

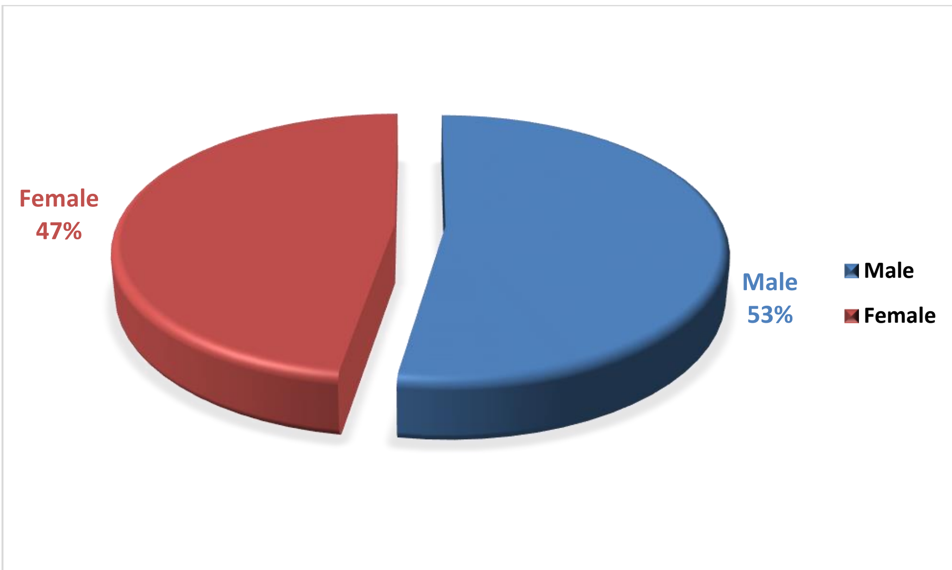


Figure -3.5: Gender of Pedestrians Interviewed

3.4.2.1.2 Interviewee`s Physical Condition

Out of 154 pedestrians who were interviewed, it was observed that only 5 pedestrians were physically disabled representing 3%. The remaining 149 pedestrians were able bodied (See Figure 3.6).

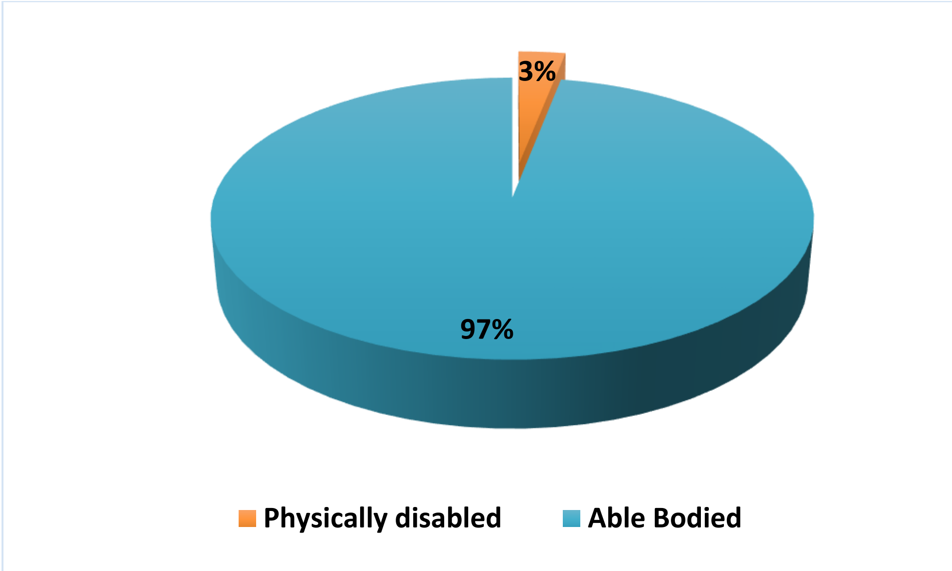


Figure -3.6: Interviewee`s Physical Condition

3.4.2.1.3 Age Group

According to Figure 3.7, the largest pedestrian age group interviewed belonged to the 17-30 years old group totaling 69% of the respondents surveyed and 28.5% of the respondents from the 31-50 years old group. The 51-66 years old group and greater than 66 years old group formed the smallest age groups with 2% and 0.5% of the respondents respectively.

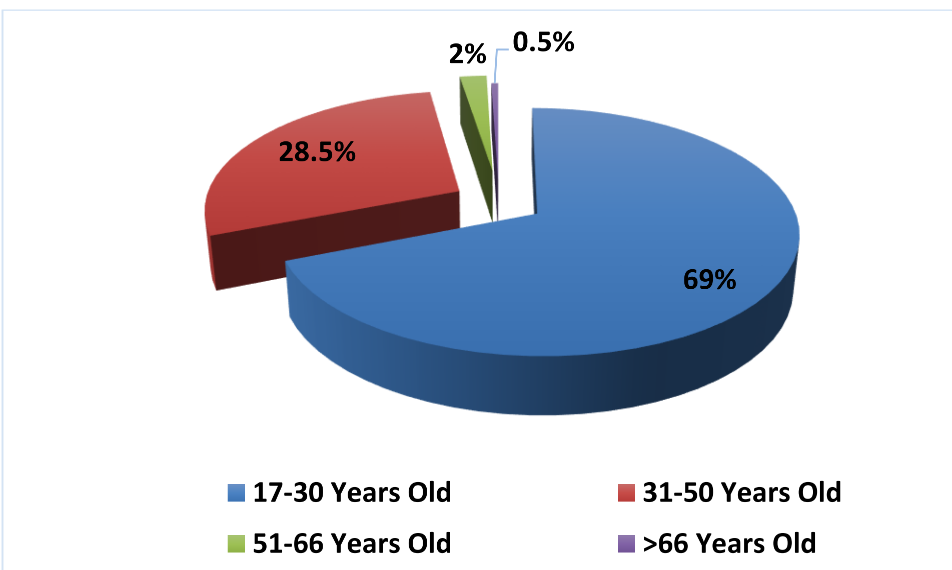


Figure -3.7: Age Structure

3.4.2.2 Origin and Destination

3.4.2.2.1Trip Purpose

Pedestrians walk from home to other designations for various purposes which include work, shopping, jolly walking, school and even business. Figure 3.8 shows that the highest percentage, 34% of the respondents crossed to return home. There is small difference in respondents who crossed the pedestrian railway station for business, education and recreation forming 27%, 22% and 17% respectively of the respondents.

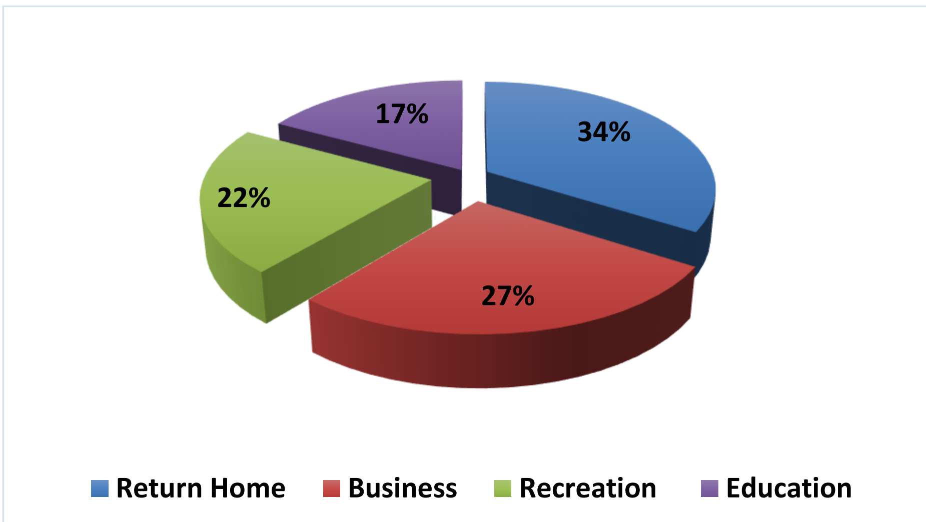


Figure -3.8: Trip Purpose for Pedestrians

3.4.2.3 Pedestrian Perception

3.4.2.3.1 Reasons for shunning pedestrian crossing on railway station

The results of the reasons why pedestrians do not want to use the pedestrian crossing on railway station.

Out of the 154 pedestrians interviewed, 114 pedestrians forming 74% of the respondents indicated that the main reason for not using the pedestrian crossing on railway station is it shunning to cross. 40 pedestrians forming 26% of the respondents felt that the pedestrian crossing on railway station is not fully utilized because it is not suitable for the physically disabled (See Figure 3.9).

During the survey conducted, it was observed that pedestrians crossed the road by jumping over the barrier instead of using the pedestrian crossing on railway station is it shunning to cross. The number of pedestrians who jay crossed the railway include figures taken both during off peak and peak hours and the number of men, females and students who opted to jump the barrier.

Figure -3.9: Pedestrian crossing railway by jumping over the barrier.

3.5 Accident Data Profile

Ethiopia has one of the highest road fatality rates in relation to vehicle ownership in the world with an average of 4 deaths from the road crashes that occur every month, according to Addis Ababa Police Commission, Bole District Community Policing Department. The detailed traffic accidents data for the past two years is indicated in the following table.

Table -3.1: Traffic accidents data for Ethiopian Fiscal years 2006 and 2007

Traffic accidents result	2006 E.C	2007 E.C
Deaths	44	57
Serious injuries	257	288
Light injuries	143	130
Asset damage	2,829	3,261
Total	3,229	3,679

Source: Addis Ababa Police Commission, Bole District Community Policing Department, Addis Ababa

Figure -3.10: Addis Ababa Ring road pedestrians climbing over the barrier.

Road Traffic Accidents exert a huge burden on Ethiopia economy in terms of hospital costs, costs of vehicle insurance and maintenance as well as indirect costs to individuals, families and society due to loss of production and effects of disability and death. In general, it is inevitable that resources which are spent on treating victims of accidents can be saved through use of underpass pedestrian crossing.

4. SOLUTION TO ADDIS ABABA LIGHT RAIL TRANSIT (LRT) DESIGN AND CONSTRUCTION PRACTICE

4.1. Planning

Pedestrian facilities have significant roles in the transport network for trips made entirely by walking and the first or last links in a trip made by other types of transport. Inadequate provision of pedestrian networks and crossing facilities can have a large impact on overall ‘connectivity’ and therefore ‘walkability’ of a route [15].

Walkability is how friendly the environment is and the ease in which pedestrians can travel through this space. It includes factors such as connectivity, legibility, safety and pedestrian Level of Service [15].

There are five general principles for planning pedestrian networks [15]:

- **Connected** – do walking networks provide good access to key destinations?
- **Comfortable** – does the path width, surface, landscaping and adjacent scale of development provide an attractive walking environment?
- **Convenient** - can streets be crossed easily, safely and without delay by all pedestrians?
- **Convivial** – are routes interesting, clean and free from threat?
- **Conspicuous** – are walking routes set out in a coherent network, clearly signposted and are they published in local maps?

Pedestrian networks should be planned to [15]:

- Minimize walking distances between land uses
- Provide a clear route to entrances of large developments (rather than surrounding car park areas)
- Avoid conflicts with vehicular movements where possible
- Provide appropriate pedestrian crossing facilities on busy roads
- Provide paths on most streets (with the exception of lightly trafficked local streets), preferably on both sides.

Pedestrian Accessibility:

Pedestrian networks should be planned in combination with land uses to provide residential access to mixed use centers and bus routes, and access to train stations of strategic and secondary activity centers [15].

Pedestrian Safety:

Pedestrian networks should be designed with passive surveillance and good lighting to provide an attractive and safe walking environment [15].

4.2. Design

4.2.1. General

The structural shape and cross section of the lining are determined by geological conditions. Underpass pedestrian crossing lines are designed for the least favourable but realistic combinations of loads and their effects on the construction materials. The design is entirely open, with the only underground walkway. It features gentle slopes and wide 10.0 meter paths, with a 4.0 meter high bridge clearance.

The design of those bridges follows the basic design principle of structural engineering that the available capacity of a structure must exceed the required capacity to support the applied loadings. The design method utilizes Load and Resistance Factor Design (LRFD) based on limit state according to ERA's Bridge Design Manual 2002, AREMA Manual and AASHTO Standard Specifications for Highway Bridges 1998.

The different limit states that needs to be checked during the design includes

- Strength limit state where the resistance of bridge structural components to bending, shear, torsion and axial loads is checked.
- Service limit state that restricts stresses deflection and crack widths under regular service conditions.
- The fatigues limit which refers to a set of restrictions on stress ranges caused by design live loads.

4.2.2. The Architectural design

The Architectural design of the underpass pedestrian crossing illustrated in the following figure was done after getting an opinion from an architect to keep its aesthetic value and its standard.

The sub-way was designed in such a way as to have entertainment rooms such as internet cafe, shops, toilets and other rooms for different purposes.

The following drawing shows, the detail partitions and their dimension in the sub-way.

Floor Plan

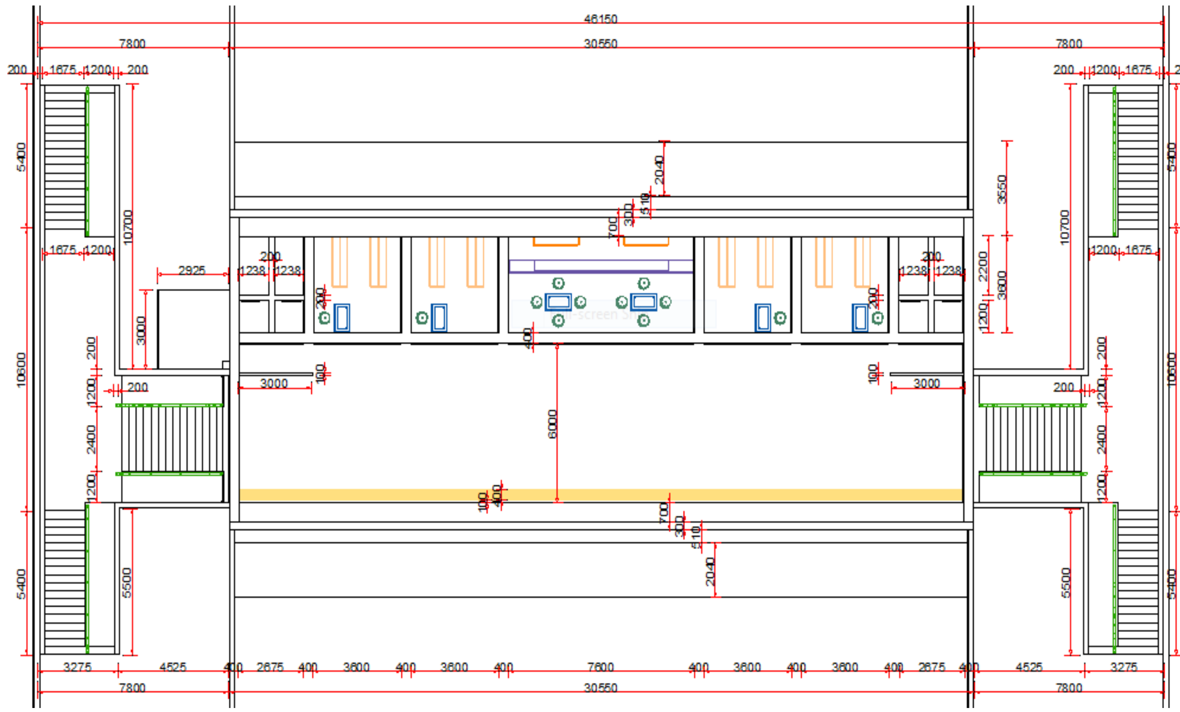


Figure -4.1: Architectural design of the pedestrian under pass crossing that connects the two sides of walkway in Addis Ababa railway (the enlarged drawing is attached in Appendix 1).

4.2.3. Design Procedure

The design of superstructure of a bridge includes design of the deck slab girder bridge and railway girder bridge. Once the span and width of the bridge is decided, material type and qualities and preliminary dimensions, which could be changed during the detailed design, should be specified.

Accordingly, the factored design loads (shear force and bending moments) were calculated at different critical sections. The sections are then designed to withstand the computed design loads. Finally, dead load and live load deflections were checked against the allowable limits and camber is provided at middle of the span as per the calculated long term dead load deflection. Like design of other structures, the design of the bridges superstructure is not one directional rather there is a possibility of going forth and back until design requirements are satisfied.

The following out line flow shows the design procedures and steps followed in the design of the bridges.

4.2.3.1. Concrete Railway Girder Bridge

In the design, the following loading conditions are considered:

- Dead loads including ballast and track ties surfacing.

- Type RU Loading (BS 5400 Section 8.2.1) Design live load for high speed railway bridges according to British standard.
  Nominal type RU Loading consists of four 250 KN concentrated loads spaced at 1.6 meter, preceded and followed by uniformly distributed load of 80 KN/m.

The following procedures are followed in the design of superstructure of the concrete Railway girders.

- Design of the deck slab bridge:

  a. Design Data and Specification:

  - Specify material properties, bridge width and spans, live loadings and design methods to be used.

  b. Bridge cross-sections and preliminary dimensions:

  - Initial preliminary dimensions are assumed which usually bases past experiences.
  - Take initial slab depths from minimum depth requirements.
  - Take girder depth, girder spacing, girder width, edge beam depth, edge beam width and initial slab depths from minimum depth requirements.
  - Take diaphragm depths and width to provide tie girders together so that can resist the load which buckling.

  c. Design the deck slab:

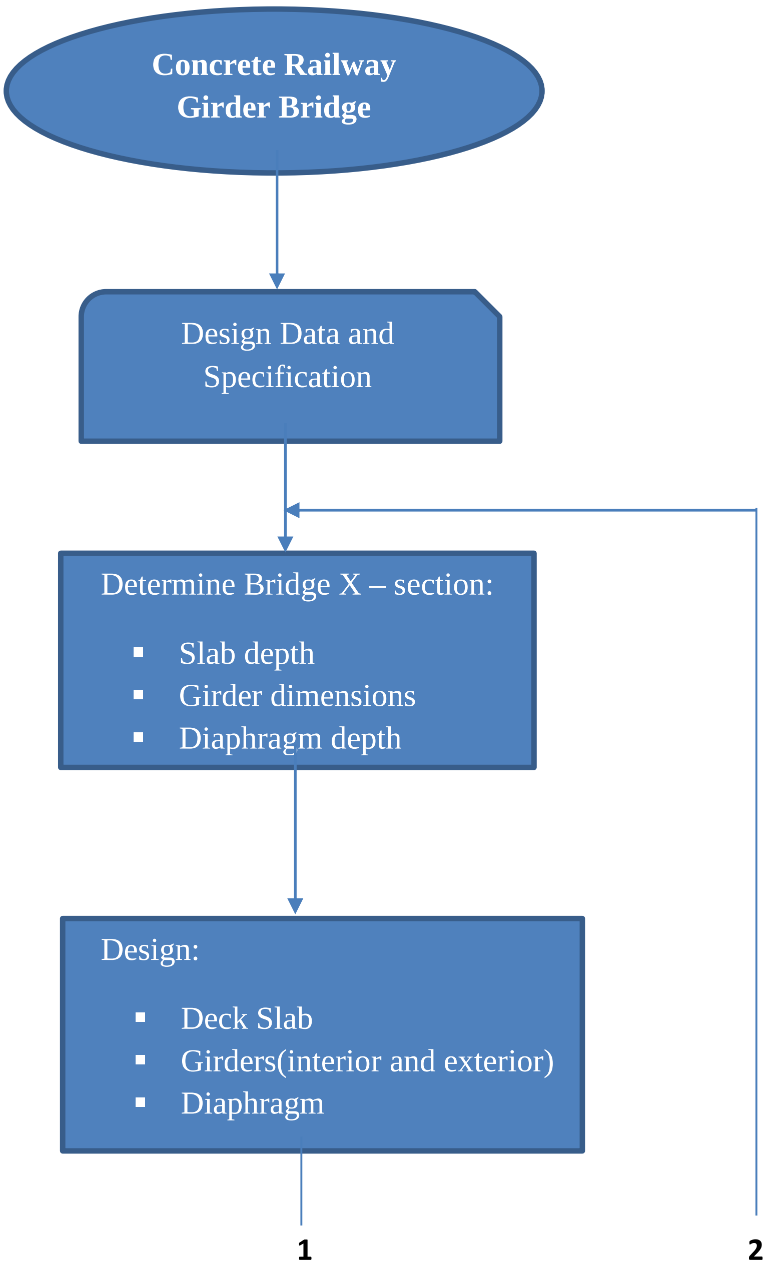
  - Determine dead load moment, live load moment from design truck and then factored design moment.
  - Provided main reinforcements for the factored design moment.
  - Check whether the requirements of minimum and maximum reinforcement are full field or not, which otherwise needs to revise section or material properties.
  - Provide distribution, temperature and shrinkage reinforcements.

- Design of longitudinal girders (interior and exterior):

  - Based on the preliminary bridge cross-sections and preliminary dimensions assumed, calculate slab dead loads and moments.
  - Determine dead load moment, live load moment from design truck and then factored design moment.
  - For transverse load distribution, specify moment and shear distribution factors.
  - Calculate factored design moments and shear.

- Design for flexure and prepare bar cutting using moment curves.
- Provided main reinforcements for the factored design moment.
- Check whether the requirements of minimum and maximum reinforcement are full field or not, which otherwise needs to revise section or material properties.
- Provide distribution, temperature and shrinkage reinforcements.

The following procedures are followed in the design of superstructure of the concrete Railway girders bridge indicated by flow chart:



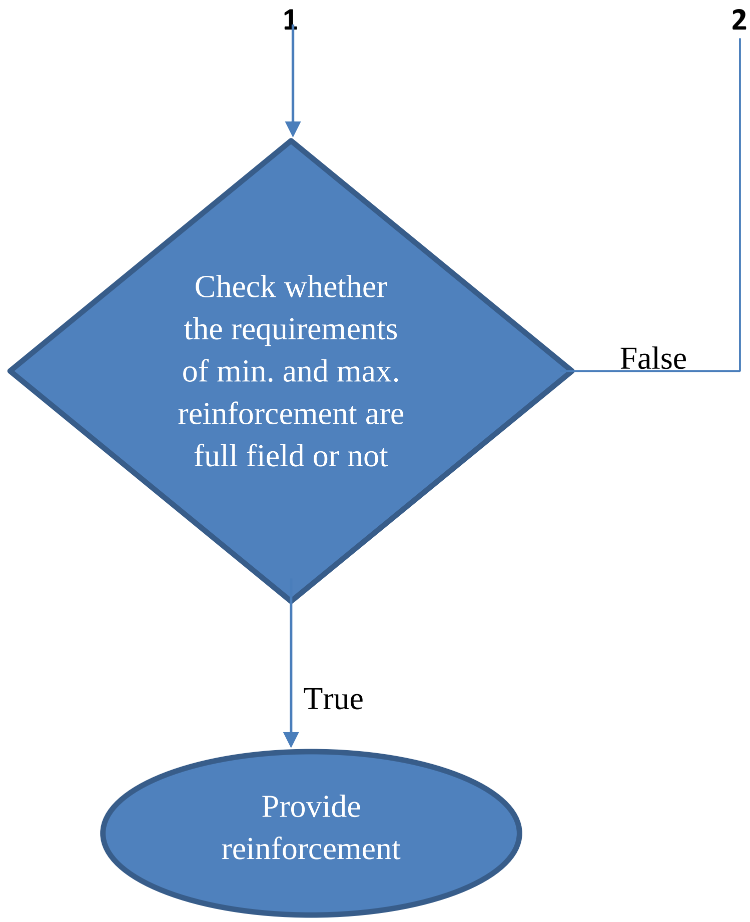


Figure -4.2: Design procedures of Concrete Railway Girder Bridge indicated by flow chart

4.2.3.2 Concrete Deck Slab Girder Bridge

In the design, the following loading conditions are considered:

- Dead loads including Asphalt surfacing
- Live load HL-93 Loading as per ERA's Bridge Design Manual (2002, 2003):

  HL-93 live load was used combined with the dead load of bridge components as per the recommended load combination of Ethiopian Road Authority (ERA) bridge design manual. HL-93 loading is a combination of truck and lane load or tandem axle and lane load and the design was performed taking the maximum effect of these loads combined with that of the dead load multiplied by the corresponding load factors.

  HL-93 live load is a combination of Design Truck load or Design Tandem load and Design Lane load. Design truck load was applied as per article 3.8.3 of ERA Bridge

Design Manual. Accordingly, three axles of the design truck the first of which is 35KN and the 2nd and the rear axles of 145KN were applied at the specified axle spacing. A total of 325KN load was used as design truckload on superstructures of bridges. In addition to the truck load a design lane load of 9.3KN/m uniformly distributed in the longitudinal direction was applied on each traffic lane of bridges. It was the combined effect of Truck and Lane load or Tandem axle and lane load whichever was greater that was taken for design of bridge components. Tandem axle loads consist of a pair of 110KN axles spaced at 1.2m intervals. The total load of tandem axle loads is far more less than that of design truck load (ERA, 2003).

The following major procedures are followed in the design of superstructure of the conventional concrete deck slab girders.

- Design of the deck slab bridge:

a. Design Data and Specification:

- Specify material properties, bridge width and spans, live loadings and design methods to be used.

b. Bridge cross-sections and preliminary dimensions:

- Initial preliminary dimensions are assumed which usually bases past experiences.
- Take girder depth, girder spacing, girder width, initial slab depths from minimum depth requirements.
- Take diaphragm depths and width to provide tie girders together so that can resist the load which buckling.

c. Design the deck slab (interior):

- Determine dead load moment, live load moment from design truck plus lane load or tandem plus lane load and then factored design moment.
- Provided main reinforcements for the factored design moment.
- Check whether the requirements of minimum and maximum reinforcement are full field or not, which otherwise needs to revise section or material properties.
- Provide distribution, temperature and shrinkage reinforcements.

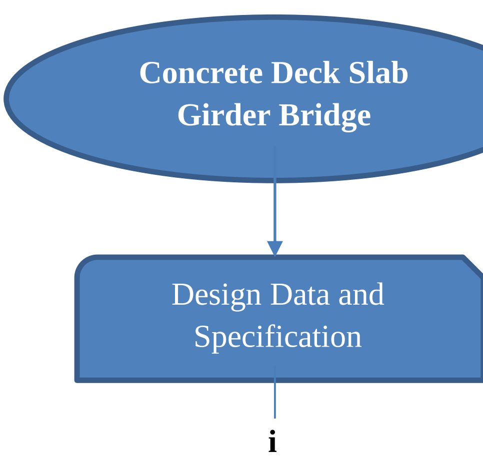
d. Design of overhang slab:

- Determine dead load moment, live load moment from design truck and then factored design moment.
- Provided main reinforcements for the factored design moment.
- Check whether the requirements of minimum and maximum reinforcement are full field or not, which otherwise needs to revise section or material properties.
- Provide distribution, temperature and shrinkage reinforcements

► Design of longitudinal girders (interior and exterior):

- Based on the preliminary bridge cross-sections and preliminary dimensions assumed, calculate slab dead loads and moments.
- Determine dead load moment, live load moment from design truck plus lane load or tandem plus lane load and then factored design moment.
- For transverse load distribution, specify moment and shear distribution factors.
- Calculate live load moments and shear which is the maximum of design truck plus lane loading or tandem load plus lane loading.
- Calculate factored design moments and shear.
- Provided main reinforcements for the factored design moment.
- Check whether the requirements of minimum and maximum reinforcement are full field or not, which otherwise needs to revise section or material properties.
- Provide camber for dead load deflection.
- Design for flexure and prepare bar cutting using moment curves.
- Provide distribution, temperature and shrinkage reinforcements.

The following procedures are followed in the design of superstructure of the Concrete Deck Slab Girder Bridge indicated by flow chart:



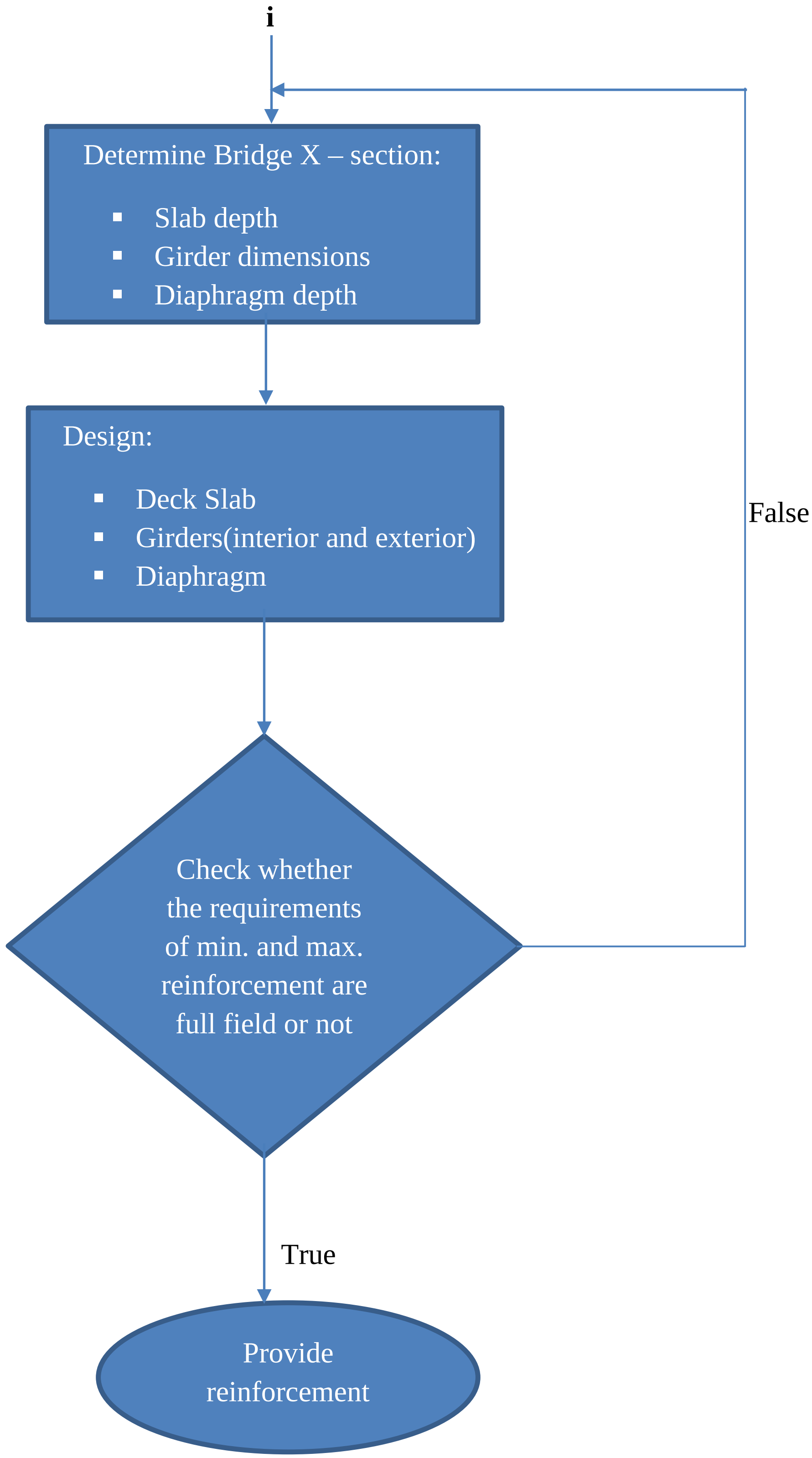


Figure -4.3: Design procedures of Concrete Deck Slab Girder Bridge indicated by flow chart

4.2.3.3 Cantilever Retaining Wall design Procedure

The following procedures are followed in the design of substructure of the cantilever retaining wall.

- Design of the cantilever retaining wall:

    a. Design Data and Specification:

    - Specify material properties.

    b. Distributed unfactored Loads from Superstructure:

    - Determine dead load, live load from Superstructure.

    c. Retaining wall cross-sections and preliminary dimensions:

    - Initial preliminary dimensions are assumed which usually bases past experiences.

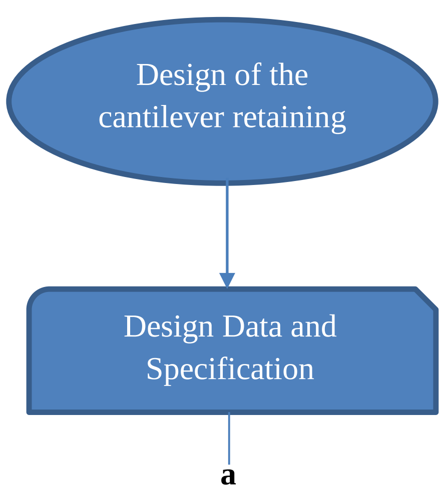
    d. Stability analysis and Lateral Loading by Rankines methods to be used:

    - Stability against Overturning
    - Stability against Sliding
    - Stability against Bearing

    e. Structural Design and Working stress design method (WSD) of the retaining walls:

    - Calculate factored design moments and shear.
    - Provided main reinforcements for the factored design moment.

The following procedures are followed in the design of substructure of the cantilever retaining wall indicated by flow chart:



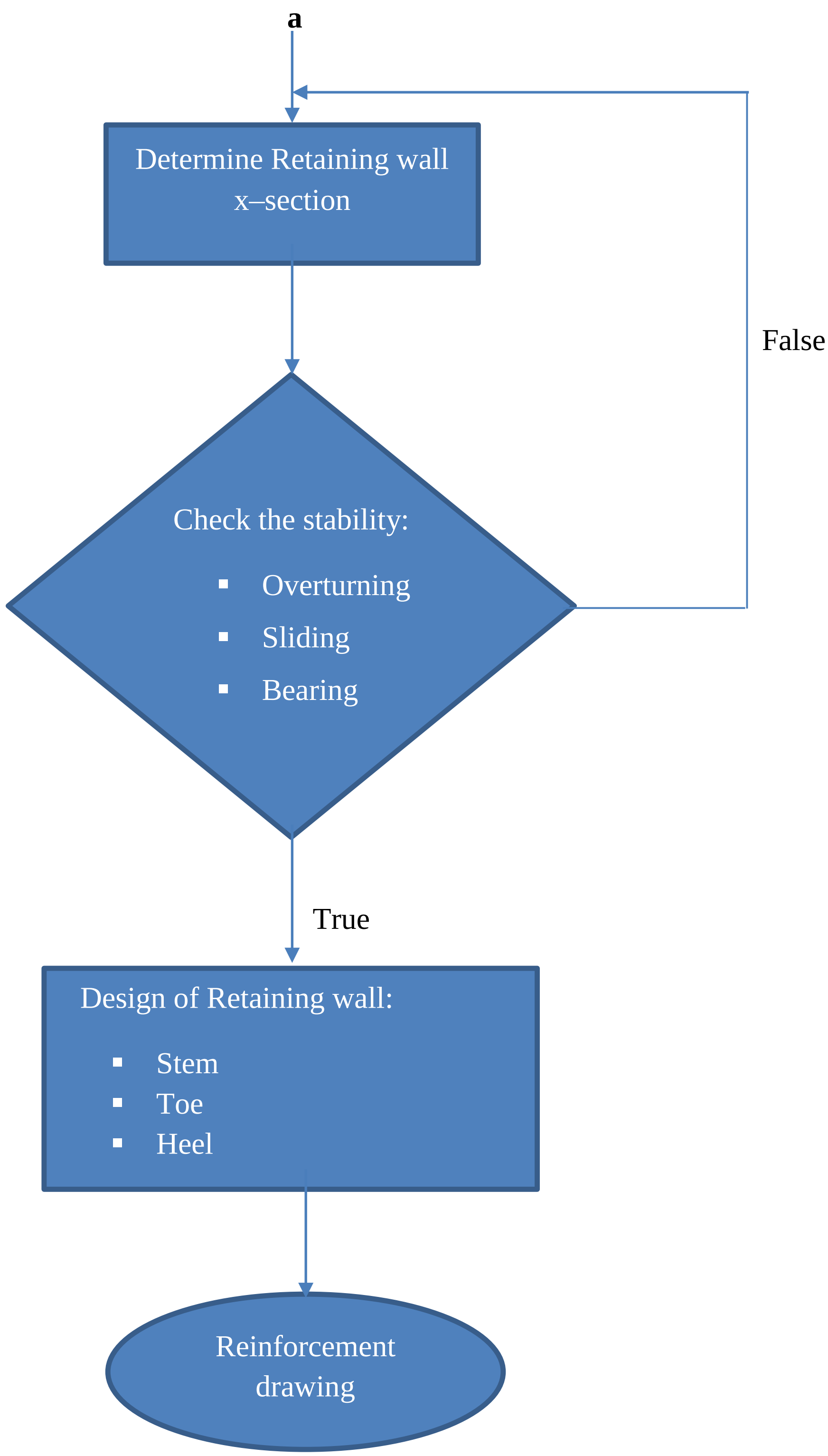


Figure -4.4: Design procedures of cantilever retaining wall indicated by flow chart

4.3. Construction and Technical specification

4.3.1. Construction methods and procedures

1) Construction methods

Construction methods for main structure: open cutting bottom-up method.

2) Main construction procedures

- Construct as protective stockade then;
- Excavate foundation pit to the first course of support, construct as the first course of support;
- Excavate foundation pit lay-by-layer to its bottom;
- Make waterproof layer for main structure roof, and fill soil from the waterproof layer to the ground.

3) Construction process of accessory structure and main structure:

Without subject to traffic organization of accessory structure could be constructed synchronized with main structure to save investment and simplify construction process.

4.3.2. Construction considerations

1) Excavation of foundation pit and construction of main structure is recommended to be conducted for underground pedestrian cross during dry season [12].

2) When excavating foundation pit, safe grade should be selected for its transverse and longitudinal slope development according to geological and environmental conditions and determined by construction unit via calculation in accordance with specific condition. Foundation pit must be excavated by sections, layers, areas and symmetrically, without over excavation. Cutting depth of each layer should not be more than 2m. It is strictly prohibited to excavate to the bottom once under one working condition [12].

3) Water diversion and discharge on the ground and in the foundation pit should be taken into account during construction. Intercepting ditch should be set on the ground around foundation pit to avoid inflow of flood and surface water into the foundation pit; after the foundation pit is excavated, underground drainage ditch and water collecting well should be set in time to prevent catchment in the pit [12].

4) Support erection should be provides in time during construction. For concrete support, excavation should be stopped whenever excavation reaches the bottom of the support. Over excavation is strictly prohibited. Every time when excavation is completed in place, construction support should be organized immediately. After the support is erected, firstly check the stability of the support and then proceed with excavation after confirmation of safety. Further excavation of the foundation pit is not allowed before strength of the support reinforced concrete reaches 80% of the design strength [12].

5) During construction, crash into the support by excavation machines is strictly prohibited; traveling of construction machines on the support is strictly prohibited. There should be no hanging machines or other loads on concrete support surface. Also, any load must not be exerted on steel support. The first course of concrete support should not be used as Construction Bridge [12].

6) During earth excavation, the waste bank should be 20m away from the foundation pit side line. There should be no loading over 20kpa within 5m outside the foundation pit. When heavy mechanical equipment operates around the foundation pit, special flat form or deep foundation should be arranged [12].

7) During construction, it is required to check in accordance with actual condition of construction and geological mapping data. Any change should be immediately reported to supervisor and design unit for adjustment on site, so as to satisfy design requirement [12].

4.3.3. Design principle of Waterproof and general rule

1) Design principle

Self-waterproof ability of concrete and reinforced concrete structures should be guaranteed. Effective technical measures should be taken to ensure the density, anti-permeability, crevice resistance, anti-corrosive property and durability of waterproof concrete specified in specifications. Waterproof measures should be intensified for structure details like deformation joints, construction joints, embedded parts, reserved holes, all kinds of joints and all kinds of structural section interfaces [12].

2) General rules

- Base slab and sidewall of main structure must be of waterproof concrete structure, and different external waterproof layer are adopted according to requirements of waterproof rates.
- Waterproof concrete of open cutting structure should also comply with the following provisions:
    a. Structure thickness should be no less than 250mm; the thickness of concrete structure at the deformation joint should be no less than 300mm.
    b. The reinforcement protection layer of upstream face should be no less than 50mm thick; the reinforcement protection layer of downstream face should be no less than 40mm thick.
- The reinforcement protection layer of upstream face should be no less than 50mm thick; the reinforcement protection layer of downstream face should be no less than 40mm thick.
- The joining parts of deformation joint, construction joint, though-wall pipe, embedded parts are vulnerable parts for waterproofing, and waterproof measures should be intensified for these parts.
- Construction of waterproof concrete at open cutting construction site must ensure water free operation. Dewatering and drainage measures should be taken when necessary to ensure that the foundation pit water level is 500mm below the cushion layer during the whole construction process.
- The dosage of waterproof concrete admixture (or addictive) should be accurate. During mixing, technician from designated admixture factory must be present to provide guidance.

- The mix proportion for construction of waterproof concrete should be determined through test. Sampling test should be conducted as required. The anti-permeability class should be one class (0.2MPa) higher than the design requirements.
- Maintenance of waterproof concrete should be conducted as required [12].

4.3.4. Technical specification

(1) The structure design should conform to the requirements of strength and rigidness checking, as well as the provision on durability, which should also meet the requirement of construction process [12].

(2) According to the engineering geology and hydrogeologic condition of the Project and requirements of urban comprehensive planning, in combination with the situation of existing buildings and road traffic, and by comprehensive comparison of technology, economy, construction process, environmental protection and service function, reasonable construction methods and structure forms should be adopted [12].

(3) The clearance size of the structure should satisfy requirements in respects of architectural clearance and other processes for application and construction, and meanwhile the construction error, measuring error and structure deformation and subsequent settlement should also be taken into consideration. In the Project, the clearance height under the bridge will be controlled [12].

(4) The structural design must be based on geological survey data. On the basis of structure and element types, working conditions and load characteristics, the structural design should apply correspondent structural design specifications and design methods in respects of the design characteristics [12].

(5) Effective measures against stray current should be taken for the structure to prevent the corrosion to structures by stray current [12].

(6) For structure design, the width of crack on concrete should be controlled, so as to prevent corrosion of enforcement bar, improve structure durability and ensure the service life of the structure [12].

(7) Design service life of the structure is 100 years [12].

(8) Ground settlement caused by construction of the Project should be put under strict control [12].

(9) Design of auxiliary structures:

- Guardrail: railing of stainless steel tubes should be adopted [12].

5. DISCUSSION

5.1. Discussion

The design of pedestrian underpass crossing that connects the two walkway sides. Different alternatives have been looked at to forward the best solution for the problem. Some of the reasons considered while forwarding the alternative solutions for the problem include the location of the area and being able to provide a lasting solution for the problem. Each alternative design for a pedestrian crossing, regardless of whether it is constructed at-grade, above grade or below grade, will have its own distinct advantages and disadvantages.

At-Grade Crossing

At-grade street crossings are the most common type of pedestrian crossings due to their relatively low-cost, design simplicity and typical location at the intersection of two streets. These crossings can be divided into two basic groups; those located at intersections or the ones at mid-block locations. Mid-block crossings tend to be less safe than crosswalks located at intersections. This is particularly true because of the simple facts that, at intersections, vehicles are usually going slower when a certain direction is given the “WALK” sign and we have become used to seeing pedestrian crossing at intersections. Conversely, mid-block crosswalk can be unexpected “inconvenience” and thereby driver reactions tend to vary more than at intersections [17].

Advantages of at grade crossing:

- ✓ Typically will have a substantially lower construction cost than either above or below grade facilities
- ✓ Construction time is much less than other crossing types. This will mean less impact on existing traffic, both pedestrian and vehicular
- ✓ Minimal or no impact on existing utility lines and drainage systems
- ✓ Easier to make connections to other transportation such as train, taxi and bus stations
- ✓ Able to incorporate a destination by adding facilities on to paths such as rest areas

Disadvantages of at grade crossing:

- ✓ Disrupts the flow of both pedestrian and vehicular traffic by having to wait for traffic signal changes
- ✓ Potential conflict will exist between pedestrians and vehicular traffic making it less safe than a grade separated crossing.

Grade Separated Crossings

Grade separation is achieved by redirecting one mode of travel either above or below the other thereby providing each with continuous channels for traffic flow [17].

When possible, working with existing topography to design an overpass or underpass will produce the most efficient structure. In addition to utilizing naturally-occurring grade changes, this method will also minimize overall costs and land-use impacts to the surrounding environment. Convenience is paramount to an effective grade separated crossing. Pedestrians will not use a poorly located crossing which takes them out of their way, or does not deliver them to a point they wish to travel. The same holds true if the crossing is perceived to be very lengthy and adds much distance to the route of travel. An overpass that is inconvenient will also be underutilized. Pedestrians may prefer a more time saving and direct option, potentially leading to hazardous conditions or creating impacts on the land [17].

Above Grade Crossings

1. Pedestrian Overpass

Pedestrian overpasses are structures that provide pedestrians, including bicyclists, a route over an obstruction such as a road. They allow for the continuous and uninterrupted flow of traffic on both the road and the path. By their nature they are typically supported on columns or other structures that elevate them over the obstacle [17].

Overpasses, particularly those crossing over high-volume roads require large vertical clearance to accommodate the various types of vehicles that may pass underneath. These structures have to be of sufficient width to accommodate two-way traffic which can include walkers, but not motorized vehicles. The width of the overpass and the associated ramps need to be essentially the same [17].

Advantages of pedestrian overpass:

- ✓ Pedestrian and vehicular traffic are channelized separately allowing traffic to move without interruption
- ✓ Because conflicts between pedestrians and vehicles are eliminated, the potential for accidents is reduced

Disadvantages of pedestrian overpass:

- ✓ An elevated, above grade structure is very expensive relative to an at-grade crossing
- ✓ Ramps connecting to the bridge take up a lot of space
- ✓ Providing direct access to Taxi and Bus stations could be difficult or impossible and could require obtaining right-of-way
- ✓ Access to some businesses could be blocked or difficult to reach
- ✓ Pedestrians may not be willing to walk the extra distance to cross the bridge.

- ✓ Height can be obtrusive into the existing urban landscape, for train electrical system and aesthetically not good.
- ✓ The overpass pedestrian crossing is not user friendly for the disabled who include those who use the clutches, wheelchairs, parents with prams and by cyclists.

2. Pedestrian Access, Crossing and Safety on the Stations

Walking is an important mode of transport to access the Bus Rapid Transit (BRT). Since pedestrians are more vulnerable to being involved in accidents, it is imperative that adequate consideration should be given to their safety through provision of facilities like guard-rails, secured crossing areas, footpaths [12].

The construction of Bus Rapid Transit (BRT) corridors provides an opportunity to build improved pedestrian footways and bicycle paths to link with trunk line stations and feeder line bus stops as part of a larger approach toward “livable cities.” Indeed, Bus Rapid Transit (BRT) systems are best viewed as part of a package of reforms promoting sustainable urban environments [12].

3. Bus Rapid Transit (BRT) and Light Rail Transit (LRT) Station Integration

Integration of LRT station with BRT station is proposed at two locations as below.

1. Near Anwar Mosque
2. Near Mexico Square

Near Anwar mosque, BRT stations are proposed adjacent to the proposed LRT station for smooth transfer. A continuous wide footpath has been proposed together with controlled crossing with table tops in the area for smooth dispersal of traffic [12].

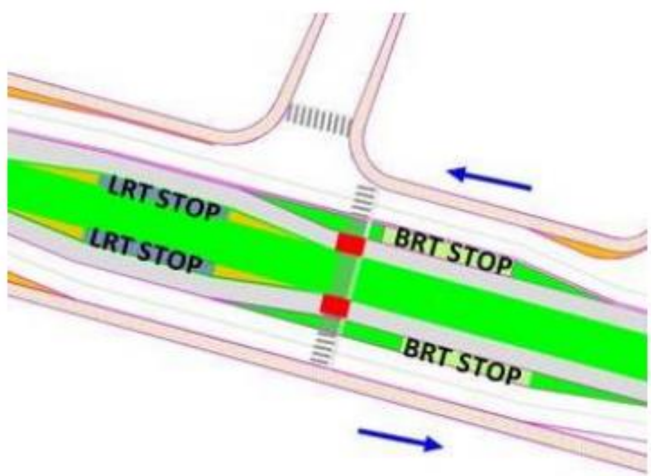


Figure -5.1: Addis Ababa Light Railway Transit Integration near Anwar Mosque Junction and near Mexico Square [12].

Figure -5.2: Addis Ababa Light Railway Transit Integration near Anwar Mosque Junction and near Mexico Square.

The BRT and LRT Stations are in line and integrated through a walkway between the stations. The passenger movement does not require vertical movement and can occur at the ground level, with the help of a table top crossing, as both the stations are at grade. The conceptual layout for integration is presented below:

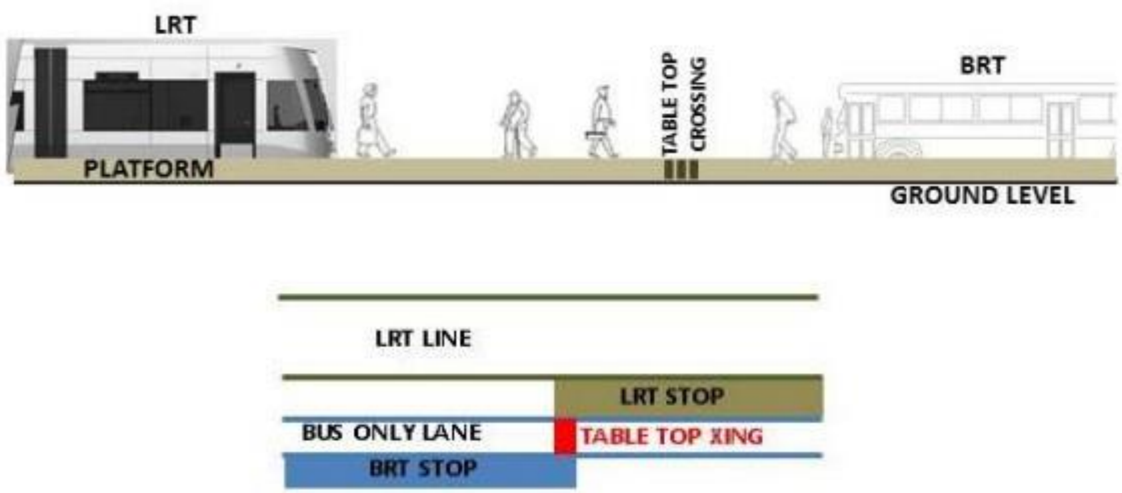


Figure -5.3: Light Railway Transit Integration with BRT [12].

Measure discussed for pedestrian access or dispersal in previous sections can be adopted for a continuous and conflict free connectivity of pedestrians to integrate BRT with nearby LRT Stops and Railway Station at Mexico Square.

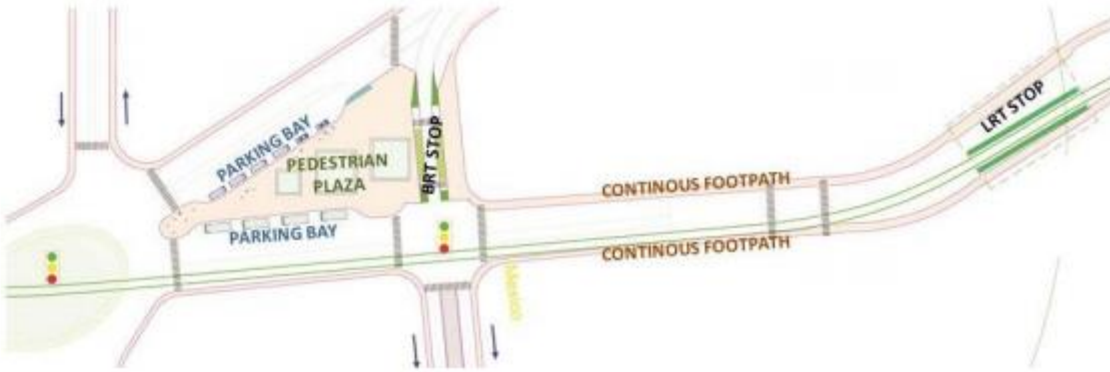


Figure -5.4: Addis Ababa Light Railway Transit Integration at Mexico Square.

When possible and good solution of the around Mexico square, but my work also good and safe solutions for Pedestrian crossing on railway. Underpass will produce the most efficient structure. In addition to utilizing naturally-occurring this method will also minimize land-use impacts to the surrounding environment.

Below Grade Crossings

1. Underpass Pedestrian crossing

Pedestrian/bicycle underpass serves many users, including bicyclists, walkers, joggers, inline skaters, and pedestrians with strollers, wheelchair users, and others. These facilities can represent one of the most important elements of a community's non-motorized transportation network. Underpasses provide critical links in the bicycle/pedestrian system by joining areas separated by a variety of "barriers." Underpasses can address real or perceived safety issues by providing users a formalized means for traversing "problem areas" such a major transportation corridors.

Underground pedestrian systems are systematic underground pedestrian spaces that have multiple functions for transport, public and commercial usage - such as underground shopping streets, and subway stations with underground concourses. Such underground pedestrian systems are normally located in sub centers and regional centers in metropolises and strongly influence city functions. Adopting a systems approach to analyzing urban pedestrian space helps to conceptualize the relationship between underground pedestrian systems and other pedestrian spaces within the broader urban environment. Firstly, viewing the city as a complex system, an underground pedestrian system is a vital subsystem of public space systems, expanding space and movement below the surface pedestrian systems. Secondly, an underground pedestrian system uses walking as a mode of transport to link and aggregate activities such as retailing and mass transit within an underground setting. Thirdly, an underground pedestrian system has an important functional feature through the provision of underground public passageways functioning as a medium that integrates ground level spaces with underground spaces. Pedestrians prefer to travel on level grade with minor undulations and very few or no major changes in elevation. A below grade crossing is an ideal way to accommodate pedestrian travel especially when the feature to be crossed is an elevated road. This can result in a below grade crossing that is actually at the same grade as the path but under the grade of the intersecting road.

In most cases, these structures are built in response to user demand for safe crossings where they previously did not exist. For instance, an underpass may be appropriate where moderate to high pedestrian/bicycle demand exists to cross a freeway in a specific location. Pedestrian/bicycle are appropriate in railroads areas where frequent or high-speed trains would create at-grade crossing safety issues. They may also be an appropriate response to railroad and other agency policies prohibiting new at-grade railroad crossings, as well as efforts to close existing at-grade crossings for efficiency, safety, and liability reasons.

The fact that this type of structure would convey pedestrian under the traffic on Addis Ababa Railway immediately brings to mind issues like 24-hour a day lighting. Proper lighting is imperative in any underpass built to improve pedestrian security. Whenever possible, any measure to improve illumination in underpass will be implemented for pedestrians to feel safer using them. In the design of the pedestrian under pass crossing that connects the two sides of walkways, besides overhead lighting natural light coming in through skylight in median opening in the ceiling from the outside was also provided (See Figure 5.4).

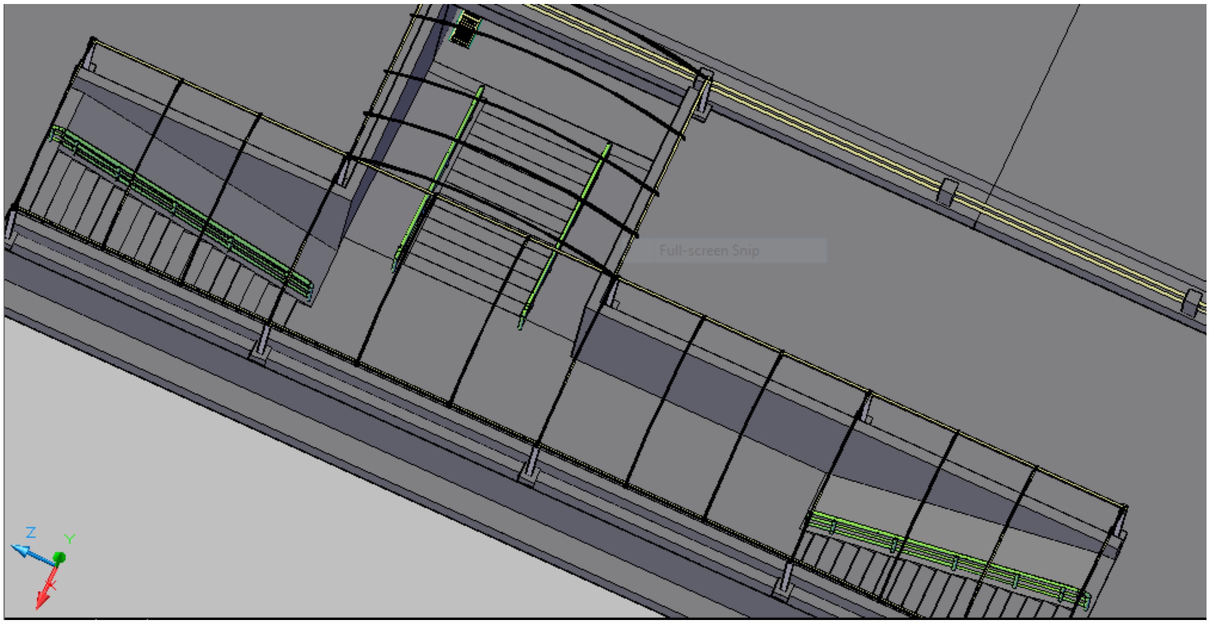


Figure -5.5: Underpass pedestrian crossing that connects the two sides of walkway on railway.

In many cases slow traffic has to cross roads for fast traffic. If there is heavy traffic flow in various directions, multi-level intersections are preferable in view of road safety and saving of time. For example a viaduct or road tunnel is very expensive, while a bridge for slow traffic crossing a speedway or railroad requires special effort to reach bridge level, which is rather high to allow sufficient headroom for the under passing fast traffic.

Actually, an underpass may be the lesser evil, indeed an underpass for slow traffic is often chosen. Therefore we have tried to develop for designers and planners a checklist of user requirements and implications are functional. People should be able to cross the road or railway safely and comfortably. Other requirements are psychological.

Closed spaces may lead to claustrophobic experiences, so measures should be taken to prevent such feelings. Furthermore, precautions are required in order to prevent feelings of insecurity e.g. fear of violence. In the next section of this article we will describe the research material on which this article is based. Subsequently we dwell on the considerations important for the optimal location of the underpass and the design objectives related to the access area.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

In conclusion, constructing underpass pedestrian crossing that connects the two sides is recommended to bring an alternative solution for the traffic problem on the mentioned street and to provide a safe way for the communities.

- Safest option for stations with poor sight distances, a high number of children, the elderly and Alcohol(drugs) impair the behaviour of pedestrians to the extent that they may be a primary cause of accident.
- Pedestrian and train(vehicular) traffic are channelized separately allowing traffic to move freely, with less interruption.
- Because conflicts between pedestrians and train (vehicles) are eliminated, the potential for accidents is reduced.
- A more direct (shortest) journey between destinations.
- Safe and convenient access for people with disabilities, parents with prams.
- The underpass is user friendly for the disabled who include those who use the wheelchairs.
- Direct crossing of the railway underpass provided Stair, ramp access and no waiting time – can cross at any time to provide protection light and security cameras of the day or night.
- Underpass pedestrian spaces that have multiple functions for transport, public and commercial usage - such as underground shopping streets, Cafeterias and subway stations with underground concourses.
- Accessibility to business, civic activities and other modes of transportation.

Hence the design of the pedestrian underpass crossing with the bridge structure over it on the Addis Ababa Light Railway Transit Project is done based on this logical ground.

6.2. Recommendations

This chapter reviews action oriented issues which must be implemented in order to improve the utilization levels of underpass pedestrian crosses. It recommends review of relevant policies that affect the use of underpass pedestrian crosses.

Basic Warrants for Provision of underpass pedestrian cross:

- Traffic calming measures should be put in places where openings are provided at underpass crossings as alternative safe crossing points. These include construction of speed humps, zebra crossing road marking and installation of road signs such as slow down and zebra crossing road signs. The responsibility lies with the road authority in charge.
- If pedestrian underpass is constructed, communities will be safe to move from one side to the other any time of the day by providing adequate illumination and by assigning guard on both the exit and entry point of the underpass.
- Space areas that will be created in the pedestrian underpass will be used for different services including internet café, secretarial services and recreational services.
- Constructing underpass pedestrian crossing to connect the two sides without any problems. These problems can be overcome by making the environment aesthetic, luxurious, providing adequate illumination along the underpass and by assigning a guard on both the exit and entry points of the underpass. Constructing underpass pedestrian cross is expensive as compared to doing at grade pedestrian pass. But this is not comparable with the loss of human life and property as a result of the traffic accident while it is easily being prevented by constructing underpass pedestrian cross.
- All the underpasses should be properly maintained in order to attract pedestrians to fully use the underpass cross as alternative safe crossing points. The responsibility lies with the road authority in charge.
- The pedestrian underpass crossing is located at Ayat – Megenegna and Kality – Meskel Square at every 300m provide pedestrian crossing.

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APPENDICES

Appendix 1 A:

A. Sample Structural Design of Railway Reinforced Concrete Bridge Superstructure

1. Design Data and Specifications

1.1 General

Subject information:

Clear Span = 10m

Road way width = 11.30m

Clear width = 10.50m

Number of Girders = 5

Span length c/c of Bearings = 10.60m

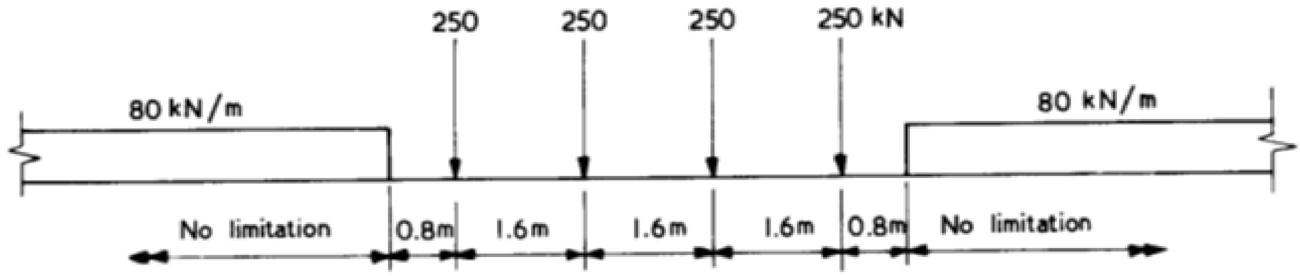
Number of Truck = 2

Face of railing to End of span = 0.40m

Effect of Multiple Truck = 1

Specification Used - AREMA Manual, AASHTO LRFD Bridge Design Spec.1998, ACI 318M-14 (March 2015) and ERA Bridge Design Manual

Loading used = Type Railway Undertaken (RU) Loading (BS 5400 Section 8.2.1) Design live load for high speed railway bridges according to British standard.



Nominal type RU Loading consists of four 250 KN concentrated loads spaced at 1.6 meter, preceded and followed by uniformly distributed load of 80 KN/m.

1.2 Material properties:

1.2.1 Concrete: - Grade C-30 concrete:

f_c'(Cylinder) = 24MPa

$f_c = 0.4 \times f_c' = 9.6$MPa

$\gamma_c(\text{density of concrete}) = 24KN/m^3$

Modulus of Elasticity $E_c = 0.043 \times (\gamma_c)^{1.5} \times \sqrt{f_c'} \times 1000$ = 24768MPa (ACI; 19.2.2.1a)

Partial safety factor, γ_c = 1.5

1.2.2 Reinforcement:

For bar Dia. $\geq$ 20mm $f_{yk} = 420Mpa$

For bar Dia. $<$ 20mm $f_{yk} = 300Mpa$

$E_s = 200000Mpa$

Partial safety factor, $\gamma_s = 1.15$

Modular Ratio, $n = E_s/E_s$ = 8.07

Take $n = 8$ Use the nearest integer

1.2.3 Track rail, inside guard rail and fastening:

Unit weight $\gamma_t = 3KN/m$

Sleeper length = 2.74m

1.2.4 Ballast including track tie:

Unit weight $\gamma_b = 19KN/m^3$

1.3 Load and Resistance Factors (LRFD) Design Methodology:

f moment = φ = 0.9

f shear = φ = 0.9

b = 0.85

Load Combination as per AREMA manual

Strength Load Design

Group-I Load= 1.4× (DL+5/3× (L+I))

Service Load Design

Group-I Load= DL+LL+I

Impact Load:

For L ≤ 4mm: I = 60 %

For < L < 39mm: I = $\frac{125}{\sqrt{L}} = \frac{125}{\sqrt{10.6}} = 38.393\%$

For L ≥ 394mm: I = 20%

Where: L= span length in 'm'.

Modulus of rupture: $f_r = 0.63\sqrt{f_c{}'} = 3.086357Mpa$ Eq. 9.50

Z in Eq.9.5 $= 30000Mpa$

For bar Dia. ≥ 20mm

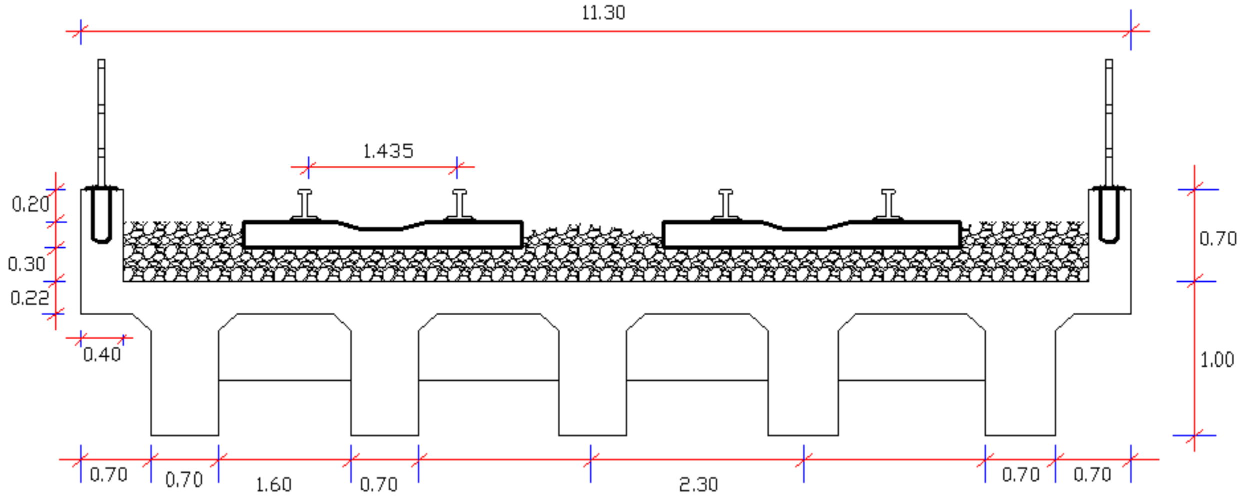
$$r_b = 0.85 \times b_1 \times {f_c{}'}/{f_c} \times \left(599.843 / (599.843 + f_y) \right) = 0.0243$$

For bar Dia. < 20mm

$$r_b = 0.85 \times b_1 \times {f_c{}'}/{f_c} \times \left(599.843 / (599.843 + f_y) \right) = 0.0385$$

$$= 0.75 \times 0.0385 = 0.0289$$

2. Preliminary Dimension



Typical Bridge Cross Section

Clear Road Way Width = 10.50m

Face of Railing to end slab = 0.40m

Total Top Width = 11.30m

Span (C/C of Bearings) = S = 10.60m

Girder Depth = $\frac{(S+2.75)}{15} = 0.89$ Take: D = 1.0m

C/C of Girder Spacing (a) = 2.30m

Overhang Distance (c) = 1.05m

Cantilever length = 1.0m

Girder web thickness = 0.70m

Clear Span between Girders = 1.60m

Top Slab Thickness (t) $= \frac{a}{12} to \frac{a}{15}$ = 0.1917m to 0.153m

Take t = 0.22

Fillets = 0.1× 0.1 b/n Girders and Deck

Minimum overhang slab Depth = L/10 = 0.105m

Take t = 0.22m

Depth of ballast (d_b) = 0.30m

Depth of tie (d_t) = 0.20m

Edge beam width = 0.40m

Edge beam H = 0.70m

Diaphragm, use at center and Ends. Thickness = 0.25m

Depth of Diaphragm = 0.60m

Total number of Diaphragm = 3

C/C distance b/n Diaphragm = 5.30m

3. Deck Slab Design

3.1 Design of Deck Slab Bridge

3.1.1 Dead Load

Slab = $\gamma_c \times t = 24KN/m^3 \times 0.22m = 5.28KN/m^2$

Ballast including track ties $= \gamma_b \times (d_b + d_t) = 19KN/m^3 \times (0.2m + 0.3m) = 9.5KN/m^2$

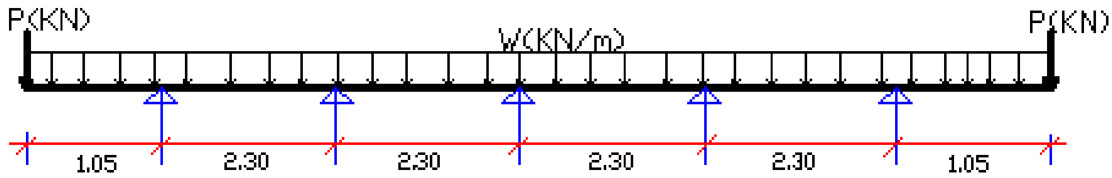
Track rail inside guard rail and fastening $= {}^{\gamma_t}/_{Sleeper\ length} = \frac{3KN/m}{2.74m} = 1.095KN/m^2$

Edge Beam = $\gamma_c \times Edge\ beam\ width \times Edge\ beam\ H\ = 24KN/m^3 \times 0.40m \times 0.70m = 6.72KN/m$

Railing Dead Load $= 0.5KN/m$

Total $D_L = 15.87\text{KN/m}^2$

Total Edge beam & railing $= 7.22\text{KN/m}$



From SAP analysis the Maximum Positive and Negative Moment

Maximum Positive moment $= 7.00KNm/m$

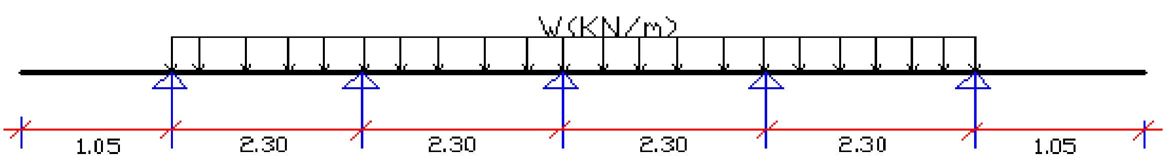
Maximum Negative moment $= 20.00KNm/m$

3.1.2 Live Load

Width of Primary strip for positive moment, $E = 660 + 0.55 \times a = 0.66 + 0.55 \times 2.3 = 1.925m$

Width of Primary strip for negative moment, $E = 1220 + 0.25 \times a = 1.22 + 0.25 \times 2.3 = 1.795m$

Live load from single track acting on the top surface of a structure with ballasted deck or under fill shall be assumed to have uniform lateral distribution over a width equal to the length of track tie plus the depth of ballast and fill below the bottom of tie, unless limited by the extent of the structure.



From SAP analysis the Maximum Positive and Negative Moment

Maximum Positive moment $= 33.00KNm/m$

Maximum Negative moment $= 59.00KNm/m$

The following results were obtained from the analysis result:

A) Only Single truck loaded	Moment	Multiple P.F	Strip width	M_{LL}(KNm/m)
$M_{max}.positive$	33	1	1.925	17.14
$M_{max}.negative$	59	1	1.795	32.87
B) Two truck loaded				
$M_{max}.positive$	0	1	1.925	0
$M_{max}.negative$	0	1	1.795	0

Summary of Maximum moments

Dead: $M_{DL-ve} = 20.00KNm/m$

$M_{DL+ve} = 7.00KNm/m$

Live Load + IM:

$$M_{LL-ve} = I \times M_{LL} = \left(\frac{38.393}{100} \times 32.87\right) = 45.50KNm/m$$

$$M_{LL+ve} = I \times M_{LL} = \left(\frac{38.393}{100} \times 17.14\right) = 23.72KNm/m$$

Factored Design Moments:

$$M_{DL-ve} = 1.4\left(DL + \frac{5}{3}(LL + I)\right) = 1.4\left(20 + \frac{5}{3}(45.5)\right) = 134.14KNm/m$$

$$M_{DL+ve} = 1.4\left(DL + \frac{5}{3}(L + I)\right) = 1.4\left(7 + \frac{5}{3}(23.72)\right) = 65.16KNm/m$$

Determination of top reinforcement:

$M_u = 134.14KNm/m$

Maximum reinforcement is limited by ductility requirement, which is given by:

Art 5.7.2.2

$a/d \leq 0.42,$ $a \leq 0.42\beta d$ $\beta = 0.85 \ for \ f_c' \leq 28Mpa$

$then: \ a = 0.357d$ $\varphi = 0.9 \quad flexure$

$M_u = 0.85\varphi f_c' \times a \times b(d - a/2) = 0.2244 f_c' b d^2$

$b = 1000mm$

$$d = \sqrt{\left(\frac{M_u}{0.2244 f_c' b}\right)} = \sqrt{\left(\frac{134.14 \times 10^6}{0.2244 \times 24 \times 1000}\right)} = 157.84mm$$

Use Ø20

Concrete cover = 50mm

$d_{provided} = t - (c.cover + \emptyset/2) = 0.22 \times 1000 - (50 + 20/2) = 160mm$ … ….***Depth is Adequate***

Reinforcement:

$$\rho = \frac{\varphi \times f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2M_u}{\varphi \times f_c'}}\right) = 0.014$$

$\varphi = 0.9\,;\ f_y = 420Mpa;\ d = 160mm\,;\ \ f_c' = 24Mpa$

$A_s = \rho bd = 0.014 \times 1000 \times 160 = 2240mm^2$

$$\rho_{min} = 0.03\frac{f_c'}{f_y} = 0.03 \times \frac{24}{420} = 0.0017\ < \rho = 0.014\ \ldots\ldots OK!$$

$$Spacing\ for\ \emptyset 20\ = \frac{\pi \times 10^2 \times 1000}{2240} = 140.24mm$$

Provide Ø20 Bars @ 140mm as top reinforcment at interior parts of deck slab

Determination of bottom reinforcement:

$M_u = 65.16KNm/m$

Maximum reinforcement is limited by ductility requirement, which is given by:

Art 5.7.2.2

$a/d \leq 0.42,$ $a \leq 0.42\beta d$ $\beta = 0.85\ \ for\ f_c' \leq 28Mpa$

$then\!:\ a = 0.357d$ $\varphi = 0.9\quad flexure$

$M_u = 0.85\varphi f_c' \times a \times b(d - a/2) = 0.2244 f_c' b d^2$

$b = 1000mm$

$$d = \sqrt{\left(\frac{M_u}{0.2244 f_c' b}\right)} = \sqrt{\left(\frac{65.16 \times 10^6}{0.2244 \times 24 \times 1000}\right)} = 110mm$$

Use Ø16

Concrete cover = 25mm

$d_{provided} = t - (c.cover + \emptyset/2) = 0.22 \times 1000 - (25 + 16/2) = 187mm \ldots \ldots$***Depth is Adequate***

Reinforcement:

$$\rho = \frac{\emptyset \times f_c'}{f_y}\left(1 - \sqrt{1 - \frac{2M_u}{\emptyset \times f_c'}}\right) = 0.006442$$

$\emptyset = 0.9\ ;\ f_y = 300Mpa; d = 187mm\ ;\ \ f_c' = 24Mpa$

$A_s = \rho bd = 0.006505 \times 1000 \times 187 = 1216.435mm^2$

$$\rho_{min} = 0.03\frac{f_c'}{f_y} = 0.03 \times \frac{24}{300} = 0.0024\ < \rho = 0.006442\ \ldots\ldots OK!$$

$$Spacing\ for\ \ \emptyset 16\ = \frac{\pi \times 8^2 \times 1000}{1216.435} = 165.28mm$$

Provide Ø16 Bars @ 160mm as bottom reinforcment at interior parts of deck slab

$A_s\ provided = 1206.37mm^2$

Distribution Reinforcement:

For Primary reinforcement Parallel to traffic:

$$S = 2.30m - 0.7m = 1.60m$$

% of Distribution reinforcement = $\frac{3840}{\sqrt{S}} \leq 67\%$

Take: % of reinforcement = 67%

$$A_s = 1206.37\left(\frac{67}{100}\right) = 808.26mm^2$$

Use Ø12

$$Spacing\ for\ \ \emptyset 12\ = \frac{\pi \times 6^2 \times 1000}{808.26} = 139.9mm$$

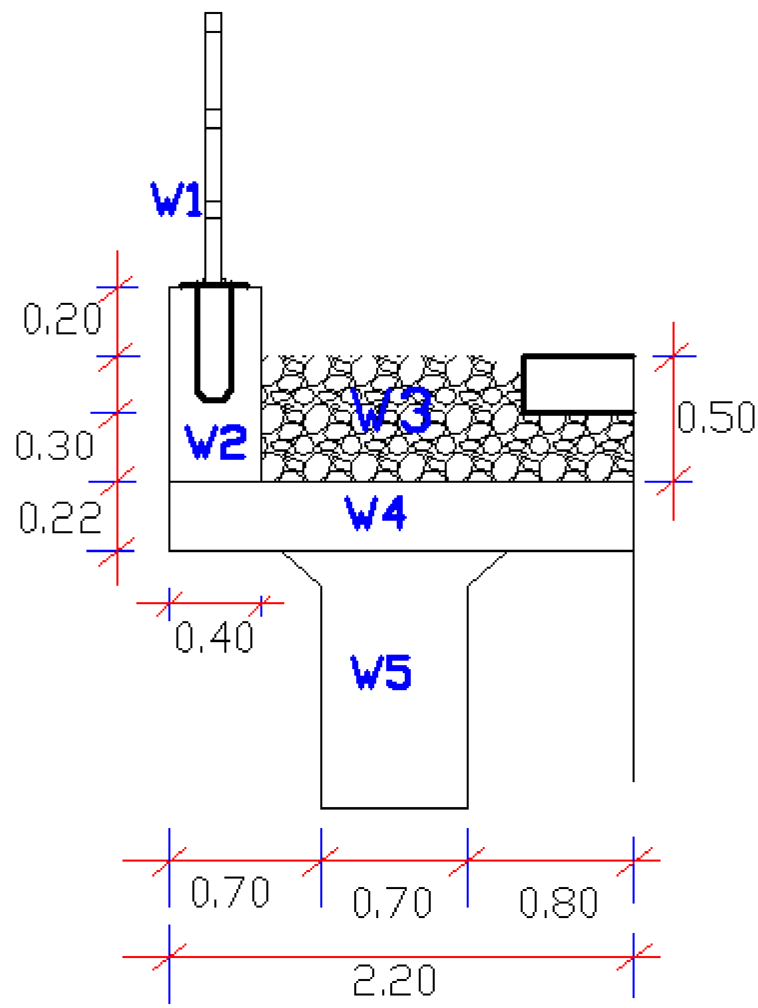
Provide Ø12 Bars @ 130mm at top and bottom slab in the Longitudinal direction reinforcement

4. Design of T-Girder

4.1 Load

4.1.1 Dead Load

A) Exterior Girder



External Girder Cross section

Dead Load per Linear meter of span:

$$\text{Flange width } = \frac{1}{2}(\text{c/c of Girder Spacing}) + \text{Overhang Distance}$$

$$= \frac{1}{2}(2.30) + 1.05 = 2.20m$$

$W_1 = 0.5KN/m$

W_2 = Edge Beam Width × Edge Beam Height × $\gamma_c = 0.4 \times 0.7 \times 24 = 6.72KN/m$

W_3 = (Depth of ballast + Depth of tie) × (Flange width −

Edge Beam Width) $\times \gamma_b = (0.3 + 0.2) \times (2.2 - 0.4) \times 19 = 17.1KN/m$

W_4 = Flange width × t × $\gamma_c = 2.2 \times 0.22 \times 24 = 11.62KN/m$

W_5 = (Girder Depth − t) × Girder web thickness × $\gamma_c = (1 - 0.22) \times 0.7 \times 24$

$= 13.104KN/m$

Fillets $= 2 \times 0.1 \times 0.1 \times 0.5 \times 24 = 0.24KN/m$

Total $= \mathbf{49.28KN/m}$

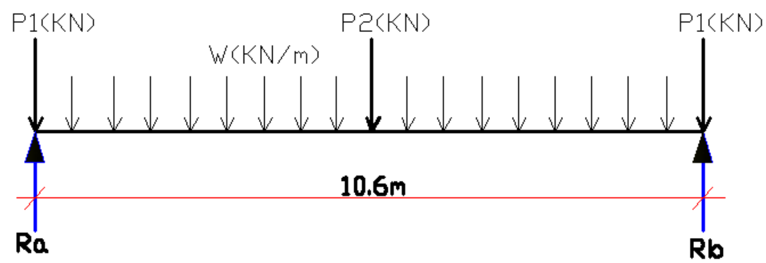
At each end = Diaphragm , use at centre and Ends thickness × (Depth of Diap@ end − t) × (c/c of Girder Spacing − Girder web thickness) × $\gamma_c \times 0.5$

$= 0.25 \times (1.3 - 0.22) \times (2.3 - 0.7) \times 24 \times 0.5 = \mathbf{5.18KN}$

W $= \mathbf{49.28KN/m}$

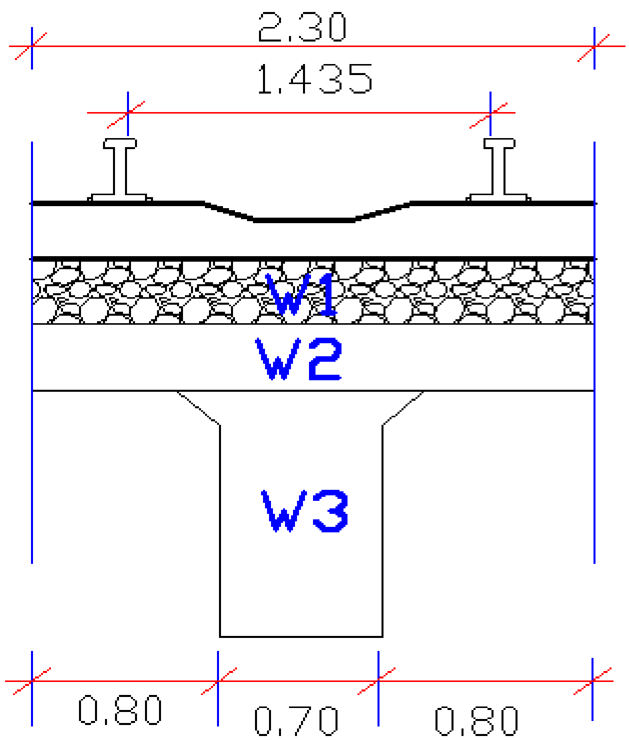
$\mathbf{P_1 = 5.18KN}$

$\mathbf{P_2 = 5.18KN}$



B) Interior Girder

Dead Load of the Highly Loaded interior girder is considered



W_1 = c/c of Girder Spacing × Edge Beam Height × γ_b

$= 2.3 \times 0.7 \times 19 = 30.59 KN/m$

W_2 = c/c of Girder Spacing × t × γ_c $= 2.3 \times 0.22 \times 24 = 12.14 KN/m$

W_3 = (Girder Depth − t) × Girder web thickness × γ_c

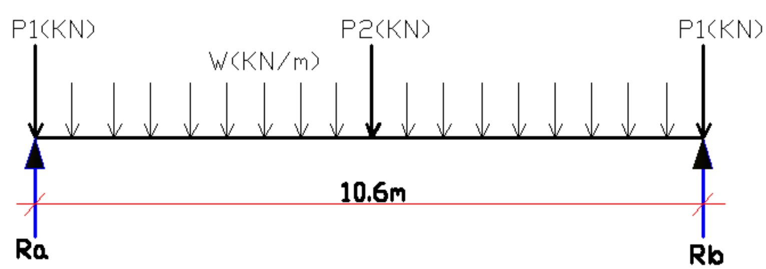
$= (1 - 0.22) \times 0.7 \times 24 = 13.104 KN/m$

Fillets $= 2 \times 0.1 \times 0.1 \times 0.5 \times 24 = 0.24 KN/m$

$\mathbf{W = 56.08 KN/m}$

$\mathbf{P_1}$ = 2 × (At each end Load) $= 2 \times 5.18 = \mathbf{10.37 KN}$

$\mathbf{P_2 = 10.37 KN}$



4.1.2 Live Load

Distribution factors for wheel load:

According to section 7.5.1 of ERA's bridge design manual live loads shall be distributed to girders based on lever rule.

A) Interior Girder

Multiple Presence factor for single track loaded = 1.0

Distribution factor including multiple presence factors shall be computed as follows.

For single track case, $\mathbf{R}_{i} = \frac{\left(1.0 \times P\left(2.3 - \left(\frac{1.435}{2}\right)\right)\right)}{2.3} = 0.688P$

<u>Summary of Transversal Distributed Loads</u>

Dead Loads:

Interior Girder: $W = 56.08KN/m$

$P_1 = 10.37KN$

$P_2 = 10.37KN$

Live Loads:

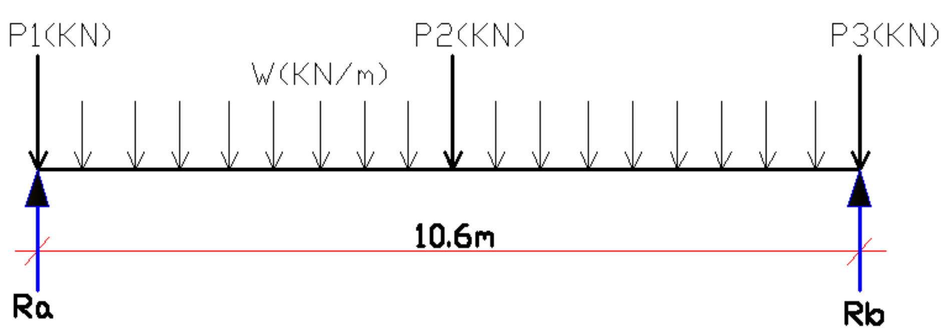
Interior Girder: Shear at support $= 0.688P$

Moment and shear in span $= 0.688P$

4.2 Analysis for Moment and Shear

4.2.1 Shear and Moment due to dead load

A) Interior Girder



$$W = 56.08KN/m$$

$$P_1 = P_3 = 10.37KN$$

$$P_2 = 10.37KN$$

$$S = 10.60m$$

$$R_a = R_b = P_1 + \frac{P_2}{2} + \frac{W \times S}{2} = 312.77KN$$

Shear at X: $V = R_a - P_1 - WX = 302.40 - 56.08X$

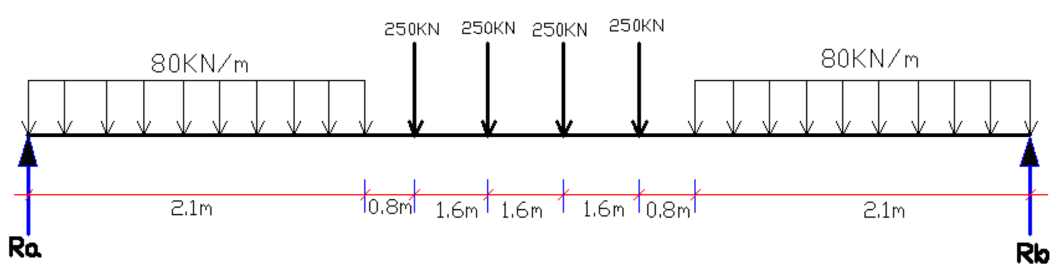
Moment at X: $R_aX - P_1X - W\frac{X^2}{2} = 302.40X - 28.04X^2$

Table -4.1: Shear and Moment due to dead load

Point	X(m)	V_{DL}(KN)	M_{DL}(KNm)	Remark
0.00	0	**302.40**	0	Maximum shear
0.05	0.53	272.68	152.39	
0.10	1.06	242.95	289.04	
0.15	1.59	213.23	409.93	
0.20	2.12	183.51	515.06	
0.25	2.65	153.79	604.45	
0.30	3.18	124.07	678.08	
0.35	3.71	94.35	735.96	
0.40	4.24	64.63	778.09	
0.45	4.77	34.91	804.47	
0.50	5.3	5.18	**815.09**	Maximum Moment

4.2.2 Shear and Moment due to live load

The maximum positive bending moment was obtained by loading with the nominal type RU loading consisting of four 250 KN concentrated loads spaced at 1.6 meter near the mid span, preceded and followed by uniformly distributed loads of 80 KN/m.



$$R_a = R_b = \frac{1}{2}(80 \times 2.1 \times 2 + 250 \times 4) = 668KN$$

$$M(x) = 668 \times x - 80 \times \frac{x^2}{2} \; for \; x \leq 2.1$$

$$M(x) = 668 \times x - 80 \times 2.1 \times (x - 1.05) \; for \; 2.1 < x \leq 2.9$$

$$M(x) = 668 \times x - 80 \times 2.1 \times (x - 1.05) - 250 \times (x - 2.9) \; for \; 2.9 < x \leq 4.5$$

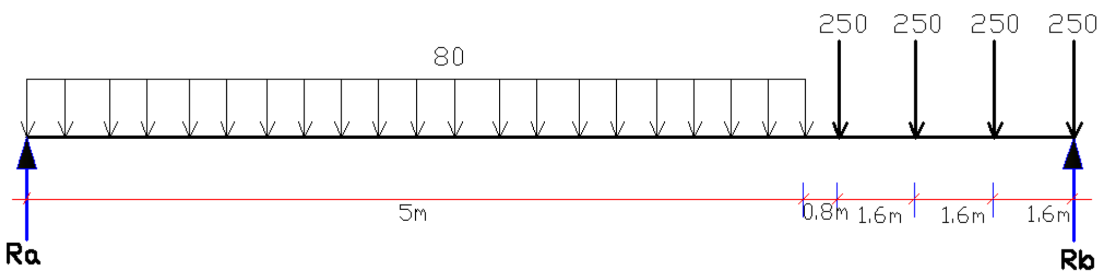
$$M(x) = 668 \times x - 80 \times 5.1 \times (x - 1.05) - 250 \times (x - 2.9) - 250 \times (x - 4.5)$$

$$for \; 4.5 < x \leq 5.3$$

Table -4.2: Moment due to live load

Point	X(m)	M_{LL}(KNm)	Remark
0.00	0	0	
0.05	0.53	342.80	
0.10	1.06	663.14	
0.15	1.59	961.00	
0.20	2.12	1236.40	
0.25	2.65	1501.40	
0.30	3.18	1696.40	
0.35	3.71	1828.90	
0.40	4.24	1961.40	
0.45	4.77	2026.40	
0.50	5.3	2026.40	

The maximum shear force is obtained by loading with the nominal type RU loading consisting of four 250 KN concentrated loads spaced at 1.6 meter near the support, followed by uniformly distributed loads of 80 KN/m.



$R_a = 867.93KN$

$R_b = 532.07KN$

Table -4.3: Shear and Moment due to live load

Point	X(m)	M_{LL}(KNm)	V_{LL}(KN)
0.00	0	0	618
0.05	0.53	342.80	618
0.10	1.06	663.14	618
0.15	1.59	961.00	618
0.20	2.12	1236.40	368
0.25	2.65	1501.40	368
0.30	3.18	1696.40	368
0.35	3.71	1828.90	118
0.40	4.24	1961.40	118
0.45	4.77	2026.40	118
0.50	5.3	2026.40	132

Maximum moment and shear due to live load considering impact.

$M_{LL} + IM = 1.3068 \times (Df \times M),$ $D_f\ interior = 0.688$

$V_{LL} + IM = 1.3068 \times (Df \times V)$

Table -4.4: Maximum moment and shear due to live load considering impact

X(m)	M_{LL}(KNm)	V_{LL}(KN)	$M_{LL} + IM$	$V_{LL} + IM$
0	0	618	0	588.43
0.53	342.80	618	326.40	588.43
1.06	663.14	618	631.40	588.43
1.59	961.00	618	915.01	588.43
2.12	1236.40	368	1177.23	350.39
2.65	1501.40	368	1429.55	350.39
3.18	1696.40	368	1615.22	350.39
3.71	1828.90	118	1741.38	112.35
4.24	1961.40	118	1867.54	112.35
4.77	2026.40	118	1929.43	112.35
5.3	2026.40	132	1929.43	125.68

4.2.3 Ultimate Maximum shear and moments

Group I Load combination = $1.4 * (\mathrm{DL} + 5/3(\mathrm{LL} + \mathrm{IM}))$ As per AREMA Manual.

Table -4.5: Ultimate Maximum shear and moments.

X(m)	$M_T(\mathrm{KNm})int.$	$V_T(\mathrm{KN})int.$	Remark
0	0	**1796.35**	Maximum shear
0.53	974.95	1754.74	
1.06	1877.92	1713.13	
1.59	2708.92	1671.52	
2.12	3467.97	1074.49	
2.65	4181.85	1032.88	
3.18	4718.17	991.27	
3.71	5093.57	394.25	
4.24	5446.92	352.64	
4.77	5628.26	311.03	
5.3	**5643.13**	300.52	Maximum Moment

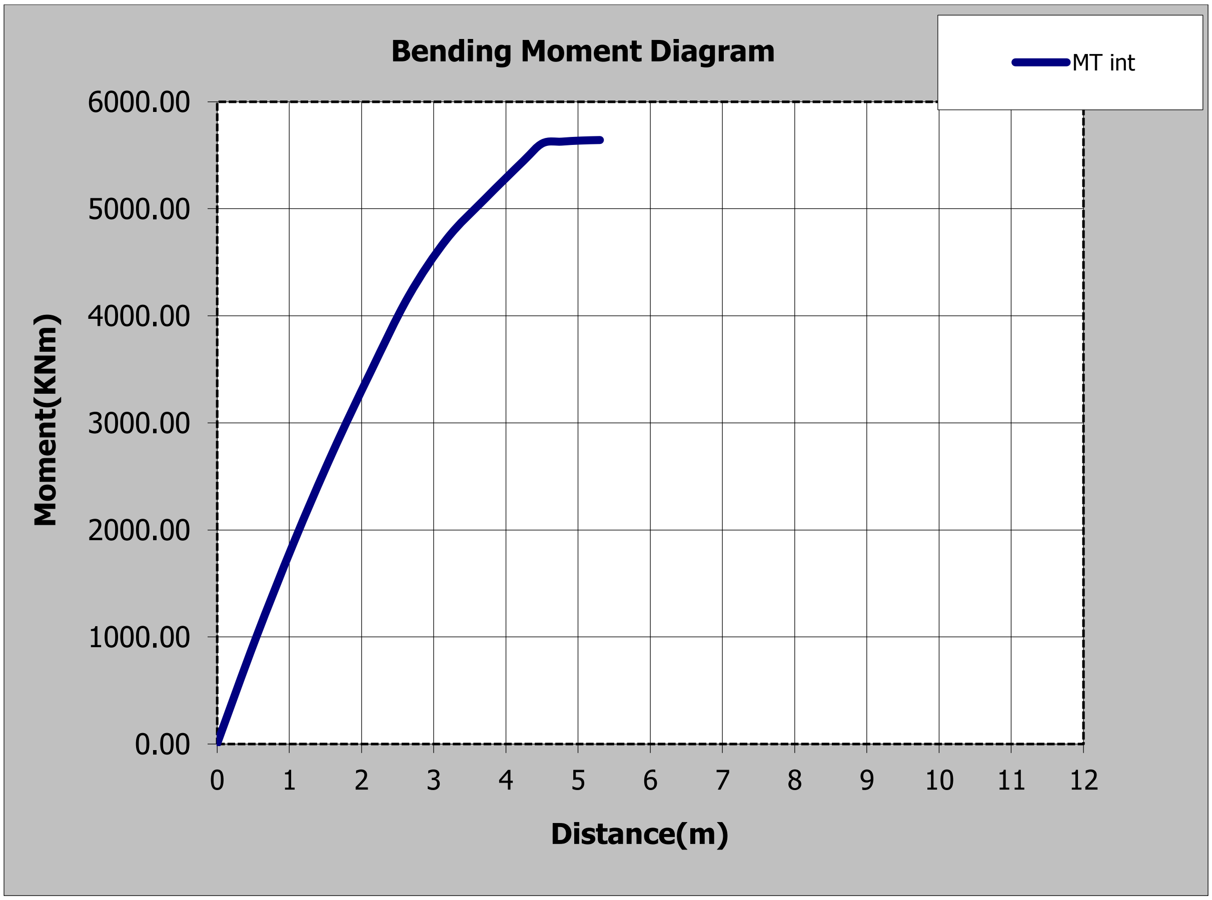


Figure -A4.1: Bending moment diagram.

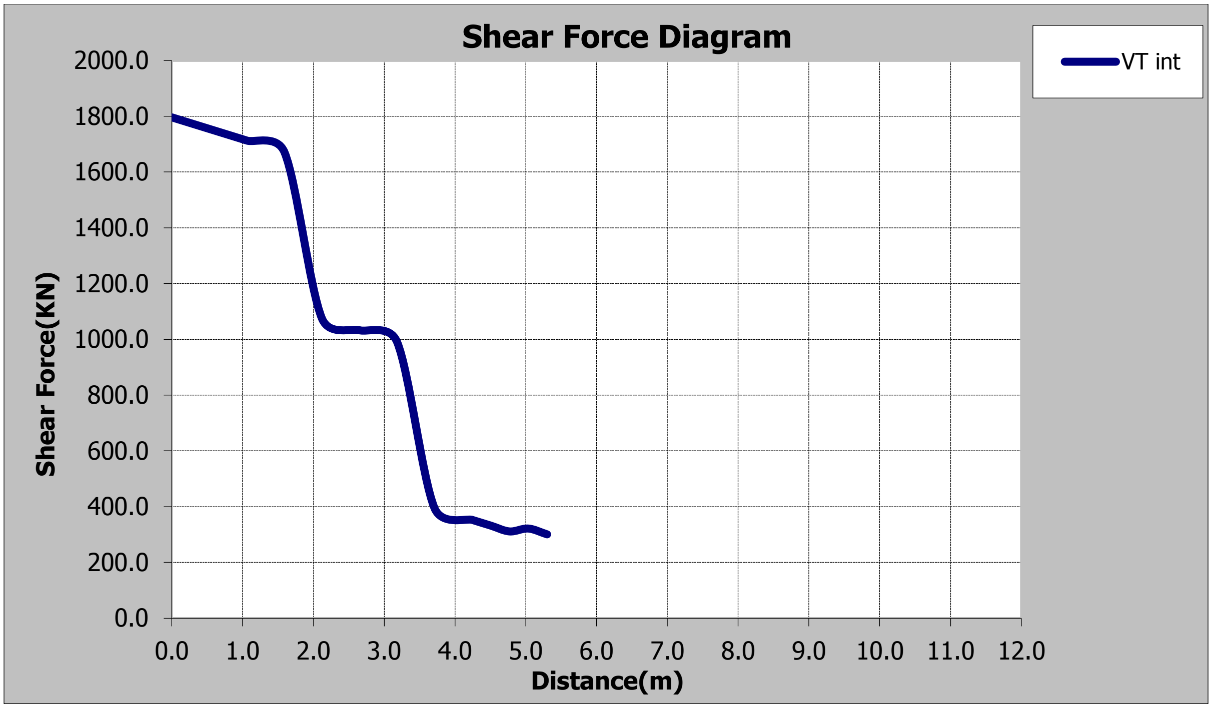


Figure -A4.2: Shear Force diagram.

5. Design of T-Girders for Bending Moment

5.1 Design Loads

The factored loads in the above tables are the design loads and the structure shall be designed to carry the expected design loads as calculated above.

5.2 Effective Compression Flange width, b_{eff} (AASHTO; Art. 4.6.2.6)

<u>Interior Girder</u>

The effective compression flange width, b_{eff} may be taken as the least of:

a) $\frac{1}{4} \times S = \frac{1}{4} \times 10.6 = 2.65m$

b) $12 \times t + (0.5 \times c/c \text{ of Girder Spacing}) = 12 \times 0.22 + 0.5 \times 2.3$
$= 3.79m$

c) Average spacing of adjacent beams $= 2.30m$

Effective Compression Flange width, $b_{eff} = min(2.65, 3.79, 2.30) = \mathbf{2.30m}$

Exterior Girder

The effective flange width may be taken as one-half the effective width of the adjacent interior beam, plus the least of the following:

a) $\frac{1}{8} \times \text{S} = \frac{1}{8} \times 10.6 = 1.33\text{m}$

b) $6 \times \text{t} + (0.5 \times \text{c/c of Girder Spacing}) = 6 \times 0.22 + 0.5 \times 2.3$
$= 2.47\text{m}$

c) $\text{Width of overhang} = 1.05\text{m}$

Effective Compression Flange width, $b_{eff} = 0.5(int.b_{eff}) + min(1.33, 2.47, 1.05)$
$= 0.5(2.3) + 1.05 = \mathbf{2.20m}$

5.3 Spacing Limits for Reinforcements (AASHTO; Art. 5.10.3)

The clear distance between parallel bars in a layer shall not be less than 1.5 times bar diameter, 1.5times maximum size of aggregate or 38mm.

Assume: $\emptyset 32$

$1.5 \times \emptyset = 1.5 \times 32 = 48mm$

$\text{Max. size of aggregate} = 1.\frac{1}{2}\text{inch} = 1.5 \times 25.4 = 38\text{mm}$

Hence the minimum c/c distance between parallel bars:

A layer with no lapping / vertical lapping is $= max(48, 38) + \emptyset = 48 + 32 = 80\text{mm}$

A layer with horizontal lapping is $= max(48, 38) + 2 \times \emptyset = 48 + 2 \times 32 = 112\text{mm}$

Except in decks where parallel reinforcing is placed in two or more layers, with clear distance between layers not exceeding 150mm, the bars in the upper layers shall be placed directly above those in the bottom layer, and the clear distance between layers shall not be less than 25mm or the nominal diameter of the bars.

Minimum clear distance b/n 2 layers of bars is 25mm $= 25mm$

Nominal diameter of bars is 32mm $= 32mm$

Thus min clear distance $= max(25, 32) = 32mm$

Hence the minimum c/c distance between parallel bars:

Multi-layer with no lapping /horizontal lapping is $= max(25, 32) + \emptyset = 32 + 32 = 64\text{mm}$

Multi-layer with vertical lapping is $= max(25, 32) + 2 \times \emptyset = 32 + 2 \times 32 = 96\text{mm}$

5.4 Section Capacity and Reinforcement

For rectangular beam analysis (a $<$ hf), use the following formulas:

$\text{r} = A_s/bd$

$\text{a} = A_s \times f_y/(0.85 \times f_c' \times b)$

$\varphi M_n = \varphi \times A_s \times f_y \times (d - a/2)$

For T-beam analysis (a $>$ hf), use the following formulas:

$r = A_s/bd$

$a = A_s \times f_y/(0.85 \times f_c' \times b)$

The steel area, A_{sf}, required to balance the longitudinal compression force in the overhang

$A_{sf} = 0.85 \times f_c' \times (b - bw) \times h_f/f_y$

The resisting moment provided by the force $A_{sf} \times f_y$

$M1 = A_{sf} \times f_y \times (d - h_f/2)$

The remaining steel area $(A_s - A_{sf})$, at a stress f_y, is balanced by the compression in the rectangular portion of the beam.

The depth of the equivalent rectangular stress block in the zone is found from the horizontal equilibrium.

$a = (A_s - A_{sf}) \times f_y/(0.85 \times f_c' \times bw)$

An additional resisting moment, M2, is thus provided by the force $(A_s - A_{sf}) \times f_y$ and $0.85 \times f_c \times bw$ at lever arm of $(d - a/2)$

$M2 = (A_s - A_{sf}) \times f_y \times (d - a/2)$

The total resisting moment, M_u, is the sum of $M1$ and $M2$

$M_u = M1 + M2$

$\varphi M_u = \varphi(M1 + M2)$

<u>Exterior Girder:</u>

The maximum design moment from the above table 4.5 is:

$M_{ultimate\ max} = 5{,}643.13KNm$

Try: Number of bar $= 40$

Diameter$(\emptyset) = 32mm$

layer	Number of bar a	$d(m)$
1^{st}	8	0.070
2^{nd}	8	0.166
3^{rd}	8	0.262
4^{th}	8	0.358
5^{th}	8	0.454
6^{th}	0	0.550
7^{th}	0	0.646

$D = 1.00m$

$$d' = \frac{sum\ a_i \times d_i}{sum\ a_i} = 0.262m$$

$d = D - d' = 1 - 0.262 = 0.738m$

$b = 2.20m$

$$A_s = 40 \times \pi \times \frac{\emptyset^2}{4} = 40 \times \pi \times \frac{32^2}{4} = 32{,}170mm^2$$
$$b = 2.20m$$
$$d = 0.738m$$
$$f_y = 420Mpa$$
$$f_c' = 24Mpa$$
$$b_w = 0.70m$$
$$h_f = 0.22m$$
$$\varphi = 0.90$$

For rectangular beam analysis (a $<$ hf), use the following formulas:

$$\mathrm{r} = \frac{A_s}{bd} = \frac{32{,}170}{2.2 \times 0.738} = 0.01981$$
$$\mathrm{a} = A_s \times f_y/(0.85 \times f_c' \times b) = 301.06mm > h_f = 220mm \;\ldots\ldots \boldsymbol{not\ Ok}!$$

For T-beam analysis are used (a $>$ hf), use the following formulas:

$$A_{sf} = 0.85 \times f_c' \times (b - bw) \times \frac{h_f}{f_y} = 16{,}025.57\mathrm{mm}^2$$
$$\mathrm{M1} = A_{sf} \times f_y \times \left(d - \frac{h_f}{2}\right) = 4{,}227.70\mathrm{KNm}$$
$$\mathrm{M2} = \left(A_s - A_{sf}\right) \times f_y \times \left(d - \frac{a}{2}\right) = 3{,}393.93\mathrm{KNm}$$
$$M_u = M1 + M2 = 7{,}621.63\mathrm{KNm}$$
$$\varphi M_u = \varphi(M1 + M2) = 0.9(7621.63) = 6{,}859.47\mathrm{KNm}$$

Therefore, the section capacity is

$$\varphi M_u = 6{,}859.47\mathrm{KNm} \quad > \quad M_{ultimate\ max} = 5{,}643.13KNm \;\ldots\ldots\ldots \boldsymbol{Ok}!$$

<u>Interior Girder:</u>

The maximum design moment from the above table is:

$$M_{ultimate\ max} = 5{,}643.13KNm$$

Try: Number of bar $= 40$

Diameter$(\emptyset) = 32mm$

layer	Number of bar a	$d(m)$
1^{st}	8	0.070
2^{nd}	8	0.166
3^{rd}	8	0.262
4^{th}	8	0.358
5^{th}	8	0.454
6^{th}	0	0.550
7^{th}	0	0.646

$D = 1.00m$

$d' = \frac{sum\ a_i \times d_i}{sum\ a_i} = 0.262m$

$d = D - d' = 1 - 0.262 = 0.738m$

$b = 2.30m$

$$A_s = 40 \times \pi \times \frac{\emptyset^2}{4} = 40 \times \pi \times \frac{32^2}{4} = 32{,}170mm^2$$

$b = 2.30m$

$d = 0.738m$

$f_y = 420Mpa$

$f_c' = 24Mpa$

$b_w = 0.70m$

$h_f = 0.22m$

$\varphi = 0.90$

For rectangular beam analysis (a $<$ hf), use the following formulas:

$\mathrm{r} = \frac{A_s}{bd} = \frac{32{,}170}{2.3 \times 0.738} = 0.01895$

$\mathrm{a} = A_s \times f_y/(0.85 \times f_c' \times b) = 287.97mm > h_f = 220mm$ ……***not Ok***!

For T-beam analysis are used (a $>$ hf), use the following formulas:

$A_{sf} = 0.85 \times f_c' \times (b - bw) \times \frac{h_f}{f_y} = 17{,}097.14\mathrm{mm}^2$

$\mathrm{M1} = A_{sf} \times f_y \times \left(d - \frac{h_f}{2}\right) = 4{,}509.54\mathrm{KNm}$

$\mathrm{M2} = \left(A_s - A_{sf}\right) \times f_y \times \left(d - \frac{a}{2}\right) = 3{,}268.73\mathrm{KNm}$

$M_u = M1 + M2 = 7{,}778.28\mathrm{KNm}$

$\varphi\ M_u = \varphi(M1 + M2) = 0.9(7778.28) = 7{,}000.45\mathrm{KNm}$

Therefore, the section capacity is

$\varphi\ M_u = 7{,}000.45\mathrm{KNm} > M_{ultimate\ max} = 5{,}643.13KNm$ ……..***Ok*** !

5.5 Checking maximum steel ratio

For rectangular sections with flexural non prestressed reinforcement only.

$\frac{a}{de} \leq 0.42$, a = distance from extreme compression fiber to the neutral axis(mm)

de = the compression effective depth(mm)

Exterior Girder:

$$a = A_s \times f_y/(0.85 \times f_c' \times b) = 301.06mm$$

$$de = D - d' = 1 - 0.262 = 0.738m = 738mm$$

$$\frac{a}{de} \leq 0.42 = \frac{301.06}{738} = 0.408 < 0.42 \ldots\ldots\ldots.\boldsymbol{Ok}!$$

Interior Girder:

$$a = A_s \times f_y/(0.85 \times f_c' \times b) = 287.97mm$$

$$de = D - d' = 1 - 0.262 = 0.738m = 738mm$$

$$\frac{a}{de} \leq 0.42 = \frac{287.97}{738} = 0.39 < 0.42 \ldots\ldots\ldots.\boldsymbol{Ok}!$$

5.6 Minimum Steel Ratio

For components containing no prestessing steel the minimum reinforcement provision shall considered satisfied if:

$$\rho_{min} = 0.03\frac{f_c'}{f_y} = 0.03 \times \frac{24}{420} = 0.001714$$

In T-beam where the web is in tension determination of steel ration shall be based on the width of the web

Exterior and Interior Girder:

Number of bars extending throughout the span $= \frac{1}{3}$ total number of bars $= \frac{1}{3} \times 40 = 14$

$$\rho = \frac{A_s}{(b_w \times d)} = \frac{14 \times \pi \times \frac{\emptyset^2}{4}}{(b_w \times d)} = \frac{14 \times \pi \times \frac{32^2}{4}}{(700 \times 738)} = 0.0218$$

$$\rho > \rho_{min} \ldots\ldots\ldots.\boldsymbol{Ok}!$$

5.7 Serviceability Requirement

5.7.1 Service Limit state (AREMA, AASHTO Manual)

Exterior Girder:

The range between the maximum and the minimum stress in straight reinforcement caused by the live load plus impact shall not.

$f_f = 145 - 0.33 \times f_{min} + 55(r/h)$, $\quad where: r/h = 0.3$

Service load factor $= 1.0$

The exterior Girder shall be checked for fatigue

$$A_s = 40 \times \pi \times \frac{\emptyset^2}{4} = 40 \times \pi \times \frac{32^2}{4} = 32{,}170mm^2$$

$b = 2.20m$

$d = 0.738m$

$r/h = 0.3$

$h_f = 0.22m$

$n = 8$

$$\rho = \frac{A_s}{(b \times d)} = 0.019814$$

$a = A_s \times f_y/(0.85 \times f_c' \times b) = 301.06mm$

$\frac{t}{d} = \frac{0.22}{0.738} = 0.2981$

$\rho n = 8 \times 0.019814 = 0.1585$

$K = \rho n + 0.5\left(\frac{t}{d}\right)^2 / \left(\rho n + \left(\frac{t}{d}\right)\right) = 0.2558$

$j = 6 - 6\left(\frac{t}{d}\right) + 2\left(\frac{t}{d}\right)^2 - \left(\frac{t}{d}\right)^3 / \left(6 - 3\left(\frac{t}{d}\right)\right) = 0.8545$

Dynamic Load allowance for fatigue limit state

$IM = 38.39\%$

$Service\ Limit\ state:\quad U = 1.00(LL + IM)$

$Factored\ Live\ L.\quad M_{(LL+I)} = 1{,}929.48$

$M_{min} = M_{DL} = 815.09KNm$

$f_{min} = \frac{M_{min}}{A_s \times j \times d} = \frac{815.09 \times 10^6}{32{,}170 \times 0.8545 \times 0.738} = 40.18N/mm^2$

$f_f = 145 - 0.33 \times f_{min} + 55\left(\frac{r}{h}\right) = 148.24N/mm^2$

$f_{(LL+I)} = \frac{M_{(LL+I)}}{A_s \times j \times d} = 95.11N/mm^2 < 148.24N/mm^2 \ldots\ldots\ldots.\boldsymbol{Ok}$!

Comment: STRESS IS WITH IN THE RANGE

Interior Girder:

The range between the maximum and the minimum stress in straight reinforcement caused by the live load plus impact shall not exceed,

$f_f = 145 - 0.33 \times f_{min} + 55(r/h),\qquad where:\ r/h = 0.3$

Service load factor = 1.0

The interior Girder shall be checked for fatigue

$A_s = 40 \times \pi \times \frac{\emptyset^2}{4} = 40 \times \pi \times \frac{32^2}{4} = 32{,}170mm^2$

$b = 2.30m$

$d = 0.738m$

$r/h = 0.3$

$h_f = 0.22m$

$n = 8$

$$\rho = \frac{A_s}{(b \times d)} = 0.01895$$

$a = A_s \times f_y/(0.85 \times f_c' \times b) = 287.97mm$

$\frac{t}{d} = \frac{0.22}{0.738} = 0.2981$

$\rho n = 8 \times 0.01895 = 0.1516$

$K = \rho n + 0.5\left(\frac{t}{d}\right)^2 / \left(\rho n + \left(\frac{t}{d}\right)\right) = 0.2504$

$j = 6 - 6\left(\frac{t}{d}\right) + 2\left(\frac{t}{d}\right)^2 - \left(\frac{t}{d}\right)^3 / \left(6 - 3\left(\frac{t}{d}\right)\right) = 0.8545$

Dynamic Load allowance for fatigue limit state

$IM = 38.39\%$

$Fatigue\ Limit\ state$: $U = 0.75(LL + IM)$

$Factored\ Live\ L.$ $M_{(LL+I)} = 1{,}929.48$

$M_{min} = M_{DL} = 815.09KNm$

$f_{min} = \frac{M_{min}}{A_s \times j \times d} = \frac{815.09 \times 10^6}{32{,}170 \times 0.8545 \times 0.738} = 40.18N/mm^2$

$f_f = 145 - 0.33 \times f_{min} + 55\left(\frac{r}{h}\right) = 148.24N/mm^2$

$f_{(LL+I)} = \frac{M_{(LL+I)}}{A_s \times j \times d} = 95.11N/mm^2 < 148.24N/mm^2 \ldots\ldots\ldots\boldsymbol{Ok}$!

Comment: STRESS IS WITH IN THE RANGE

5.7.2 Control of Cracking by Distribution of Reinforcement (Sec. 9.4.5)

Exterior Girder: (ERA; Equ. 9.14)

To control flexural cracking of the concrete, tension reinforcement shall be well distributed within maximum flexural zones.

Components shall be so proportioned that the tensile stress in the steel reinforcement at service limit state, f_s does not exceed.

$y' = d' = 0.262m$

$b_w = 0.70m$

$N_o of\ bars = 40$

$f_s = \frac{Z}{(dc \times A)^{1/3}} \leq 0.6 \times fy$

$$A = \frac{Effective\ tension\ area}{N_o of\ bars} = \frac{bw \times (y')}{N_o} = \frac{0.7 \times (0.262)}{40} = 0.00459m^2$$

dc = distance measured from extreme tension fiber to center of the closest bar $= 0.07m$

$Z = crack\ width\ parameter, assumed = 23KN/mm$

Therefore, for crack control the maximum allowable stress is

$$f_s = \frac{Z}{(dc \times A)^{\frac{1}{3}}} = \frac{23}{(0.07 \times 0.00459)^{\frac{1}{3}}} = 335.8N/mm^2$$

$0.6 \times fy = 0.6 \times 420 = 252N/mm^2$

The maximim stress, fmax, at service load is

$S_{max} = (M_{DL} + M_{LL} + IM) / (As \times j \times d) = 135.29N/mm^2 < 252N/mm^2$ …. $\boldsymbol{Ok}$!

Interior Girder:

To control flexural cracking of the concrete, tension reinforcement shall be well distributed within maximum flexural zones.

Components shall be so proportioned that the tensile stress in the steel reinforcement at service limit state, f_s, does not exceed.

$$y' = d' = 0.262m$$
$$b_w = 0.70m$$
$$N_o of\ bars = 40$$

$$f_s = \frac{Z}{(dc \times A)^{1/3}} \leq 0.6 \times fy$$

$$A = \frac{Effective\ tension\ area}{N_o of\ bars} = \frac{bw \times (y')}{N_o} = \frac{0.7 \times (0.262)}{40} = 0.00459m^2$$

dc = distance measured from extreme tension fiber to center of the closest bar $= 0.07m$

$Z = crack\ width\ parameter, assumed = 23KN/mm$

Therefore, for crack control the maximum allowable stress is

$$f_s = \frac{Z}{(dc \times A)^{\frac{1}{3}}} = \frac{23}{(0.07 \times 0.00459)^{\frac{1}{3}}} = 335.8N/mm^2$$

$0.6 \times fy = 0.6 \times 420 = 252N/mm^2$

The maximim stress, fmax, at service load is

$S_{max} = (M_{DL} + M_{LL+IM}) / (As \times j \times d) = 135.29N/mm^2 < 252N/mm^2$ …. $\boldsymbol{Ok}$!

5.8 Bar cutting and Resisting Moment

At least one third the positive moment reinforcement should be extended in to supports

For exterior girder $= \frac{N_o}{3} = \frac{40}{3} = 13.33 \quad extend \ = 16\text{bars}$

For interior girder $= \frac{N_o}{3} = \frac{40}{3} = 13.33 \quad extend \ = 16\text{bars}$

<u>Development of Reinforcement</u> (ERA Sec. 9.4.3)

Positive moment reinforcement: At least one-third of the positive moment reinforcement in simple span members shall extend along the same face of the member beyond the centreline of the support.

Reinforcement shall be extended beyond it is no longer needed to resist flexure for a length equal to the maximum of the followings.

The effective depth of the members , $d = 738\text{mm}$

20 times the nominal diameter of the bar , $d = 20 \times \emptyset = 20 \times 32 = 640\text{mm}$

0.0625 of the clear span $= 0.0625 \times 10.6 \times 1000 = 662.5mm$

Take extended length of bar $= \max(738, 640, 662.5) = 738mm$

- Continuing reinforcement shall extend not less than the development length, l_d, (ERA Sec. 9.4.5)

The basic development length, l_d, in mm

For bars diam. 32 and smaller, $\text{l}_d\text{b} \ = \ 0.02\text{Ab} \times \text{f}_y/\sqrt{\text{f}_c{}'}$

$\text{f}_y = 420Mpa$

$\text{f}_c{}' = 24Mpa$

$\text{modification factor} = 2.0$

A_b = minimum area of bar (mm^2)

$$\text{l}_d\text{b} \ = \ 0.02\text{A}_b \times \frac{\text{f}_y}{\sqrt{\text{f}_c{}'}} = 0.02 \times \left(\pi \times \frac{32^2}{4}\right) \times \frac{420}{\sqrt{24}} = 1379mm$$

The tension development length, l_d, is

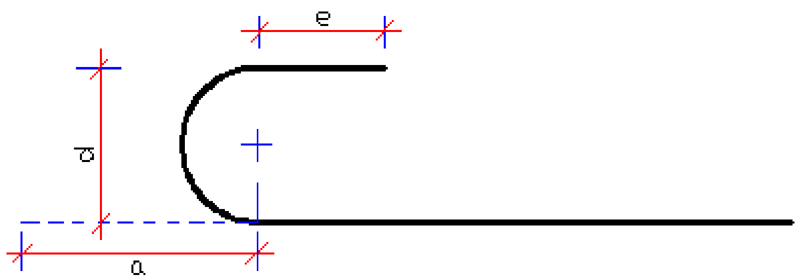
$\text{l}_d = \text{l}_d b \times \text{modification factor} = 1379 \times 2.0 = 2758\text{mm} = 2.76\text{m}$

Lap splices of Reinforcement in Tension

The length of lap for tension lap splices shall not be less than either 300mm or the 1.3 times the development length.

$\text{Lap splices for diam. 32 bar} = \max(1.3 \times l_d, 300) = \max(3588, 300) = \mathbf{3588mm}$

Standard Hooked-Bar



$$e = 4 \times \emptyset = 4 \times 32 = 128mm$$

$$d = 10 \times \emptyset = 10 \times 32 = 320mm$$

$$a = \frac{1}{2} \times circumf. + 4 \times \emptyset = \frac{1}{2} \times \pi d + 4 \times 32 = 630mm$$

Length of each bar for flexural reinforcement is determined below:

No of bars	$A_s(m^2)$	$d'(ext)$	$d'(int)$	$deff\_ext$	$deff\_int$	$a(ext)$	$a(int)$	$M_{cap(ex)}$	$M_{cap(int)}$
40	0.0321699	0.256	0.256	0.744	0.744	301.1	288.0	7216.76	7296.34
32	0.0257359	0.209	0.209	0.791	0.791	240.8	230.4	6523.50	6574.44
24	0.0193019	0.162	0.162	0.838	0.838	150.6	172.8	5455.20	5485.85
16	0.0125679	0.115	0.115	0.885	0.885	120.4	115.2	4011.85	4024.58
8	0.0064339	0.070	0.070	0.930	0.930	60.2	57.6	2188.58	2191.77

Exterior Girder (as critical): Total Girder length $= 10.60m$

$x(m)$	Actual Moment(KNm)	Type	No of bars	Total No. of bars	Resisting Moment	Cutting Moment
0.000	0.00	G00	16	16	4011.85	**No cutting**
0.530	974.95	G11	8	24	5455.20	4011.85
1.060	1877.92	G22	8	32	6523.50	5455.20
1.590	2708.92	G33	8	40	7216.76	6523.50
2.120	3467.97					
2.650	4181.85					
3.180	4718.17					
3.710	5093.57					
4.240	5446.92					
4.505	5612.55					
4.770	5628.26					
5.035	5638.45					
5.300	5643.13					

Length of Flexural Reinforcement Bar	
Bar - G1 = 16* Diam. 32 bars	11.26m
Bar - G2 = 8* Diam. 32 bars	8.60m
Bar - G3 = 8* Diam. 32 bars	6.80m
Bar - G4 = 8*Diam.32 bars	6.00m

5.9 Skin reinforcement

If the depth of side face of a member exceeds 3ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the member for a distance $\frac{d}{2}$ nearest the flexural tension reinforcement.

Area of skin reinforcement, $A_{sk} \geq 0.012 \times (d - 30)\text{in}^2/\text{ft}$ (on each side face)

$$d = 0.738m$$

$$\emptyset = 16mm$$

$$A_{sk} = 0.012 \times \left(\left(\frac{0.738 \times 100}{2.54}\right) - 30\right)\text{in}^2/\text{ft} = -0.01134\text{in}^2/\text{ft} = -24\text{mm}^2/m$$

$$Spacing = \frac{as}{A_s} \times 1000 = -8378mm$$

$Maximum\ spacing, Smax, lesser\ of\ d/6\ or\ 12"$ $= (0.738 \times 1000/6, 12 \times 25.4)$

$= (123, 304.8) = 123mm$

$$\boldsymbol{Use\quad \emptyset 16\ c/c\quad -8370mm}$$

6. Design of Girder for shear

Nominal shear resistance V_n is determined as:(AASHTO 5.8.3.3)

Correction factor for skewness = 1.0

$$V_n = V_c + V_s$$

$$V_c = 0.083B \times b_v \times d_v \times \sqrt{f_c{}'}$$

$$b_v = web\ effective\ width\ = 0.7m$$

$$d_v = effective\ shear\ depth\ = 0.738m$$

$$\beta = factor\ of\ ablity\ of\ section\ to\ resist\ tension = 2.0(traditional\ value)$$

$$\theta = angle\ of\ inclination\ of\ diagonal\ compressive\ stresses = 45°$$

$$V_c = 0.7 \times 0.738 \times 0.083 \times 2 \times \sqrt{24} = 420.11KN$$

$$\varphi V_c = 420.11 \times 0.9 = 378.1KN$$

$$V_s = A_v \times f_y \times \frac{d_v}{S} \times cot\theta,$$

$If\ diameter\ 12\ bar\ are\ used$

$$A_v = 2A_s = 2\frac{\pi Ø^2}{4} = 2\frac{\pi 12^2}{4} = 226.19\text{mm}^2$$

$$S = A_v \times f_y \times \frac{d_v}{V_s} \times cot\theta = 50.08/V_s$$

$$V_s = V_u - \varphi V_c$$

Minimum transverse reinforcement

$$A_v = 0.083 \times b_v \times S/\left(f_y \times \sqrt{f_c{}'}\right)$$

$$Max.spacing = S = A_v \times f_y/\left(0.083 \times b_v \times \sqrt{f_c{}'}\right)$$

$$S_{max} = 238.4mm$$

Maximum spacing for traverse reinforcement

If $V_u < 0.1 \times f_c' \times b_v \times d_v$, $\quad then\ \ S \leq 0.8 \times d_v = 0.8 \times 738 = 590.4mm$

If $V_u \geq 0.1 \times f_c' \times b_v \times d_v$, $\quad then\ \ S \leq 0.4 \times d_v = 0.4 \times 738 = 295.2mm$

$$= 0.1 \times f_c' \times b_v \times d_v = 0.1 \times 24 \times 0.7 \times 0.738 = 1239.84\text{KN}$$

Therefore maximum spacing shall be 23 cm

Shear reinforcement ranges for the sections of the girder is computed as follows

The maximum shear forces of the girder are used for design

Correction factor for shear due to obtuse support $= r = 1.0 + 0.2 \times (Lt^3/Kg)tan\theta = 1.0$

X	V_u	V_s	$S_{req}(mm)$	S_{max}	$S_{provided}$
0	1796.35	1418.25	35.3	295.20	6cm
0.53	1754.74	1376.64	36.4	295.20	6cm
1.06	1713.13	1335.03	37.5	295.20	6cm
1.59	1671.52	1293.42	38.7	295.20	6cm
2.12	1074.49	696.39	71.9	238.41	10cm
2.65	1032.88	654.78	76.5	238.41	10cm
3.18	991.27	613.17	81.7	238.41	10cm
3.71	394.25	16.14	3102.5	238.41	10cm
4.24	352.64	-25.47	-1966.3	238.41	No
4.505	331.83	-46.27	-1082.3	238.41	No
4.77	311.03	-67.08	-746.6	238.41	No
5.035	321.32	-56.78	-882.0	238.41	No
5.3	300.52	-77.58	-645.5	238.41	No

Appendix 1 B:

B. Sample Structural Design of Reinforced Concrete Girder Bridge Superstructure

1. Design Data and Specifications

1.1 General

Road way width = 7.40m

Curb width = 1.10m

Total width = 9.60m

Clear Span = 10m

Span length c/c of Bearings = 10.60m

Total Span length/Superstructure/ = 11.20m

1.2 Material properties:

1.2.1 Concrete: - Grade C-30 concrete:

f_c'(Cylinder) $= 24$MPa

$f_c = 0.4 \times f_c' = 9.6$ MPa

γ_c(density of concrete) $= 24KN/m^3$

Modulus of Elasticity $E_c = 0.043 \times (\gamma_c)^{1.5} \times \sqrt{f_c'} \times 1000$ = 24768MPa (ACI; 19.2.2.1a)

Partial safety factor, γ_c = 1.5

$\varphi = 0.9$

1.2.2 Reinforcement:

For bar Dia. $\geq$ 20mm $f_{yk} = 420Mpa$

For bar Dia. $<$ 20mm $f_{yk} = 300Mpa$

$E_s = 200000Mpa$

Partial safety factor, $\gamma_s = 1.15$

Modular Ratio, $n = E_s/E_s$ = 8.07

Take $n = 8$ Use the nearest integer

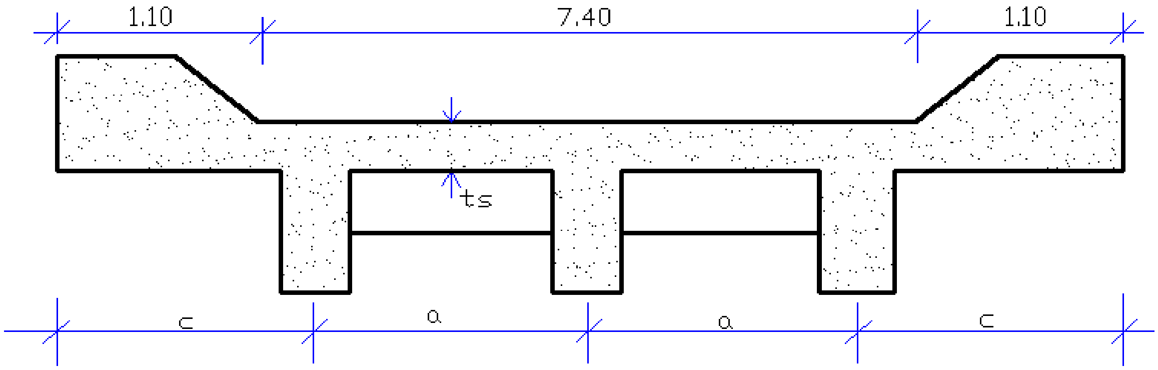
Design Method = Load and Resistance Factors Design (LRFD):

Reference: ERA's Bridge Design Manual 2002

ERA's Standard Technical Specifications 2002

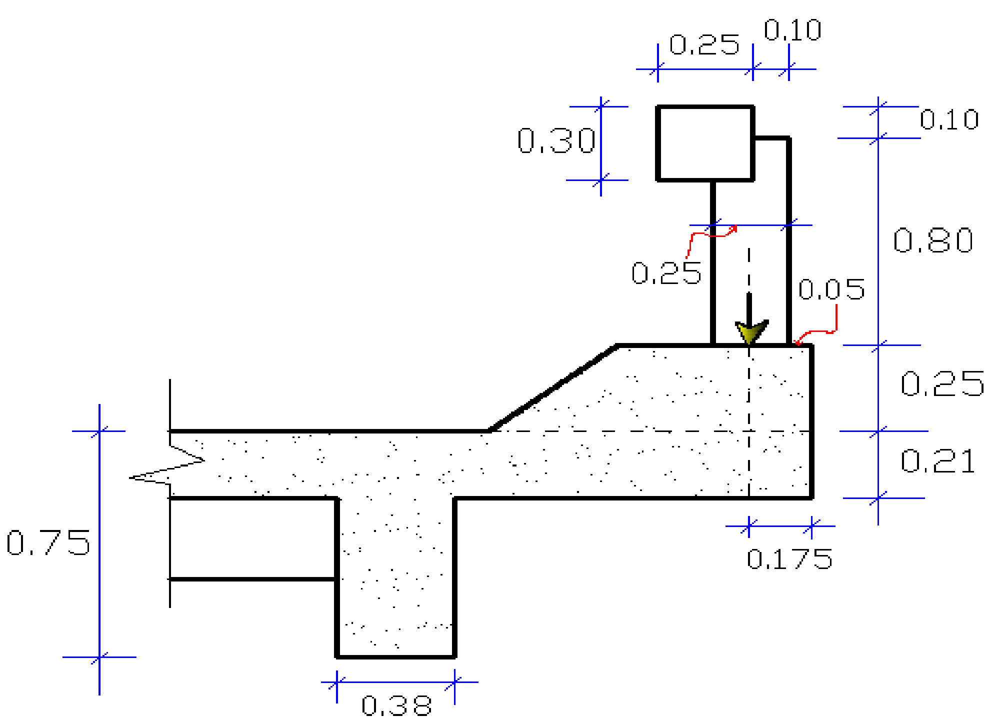
AASHTO LRFD Bridge Design Specifications, 1998 Edition.

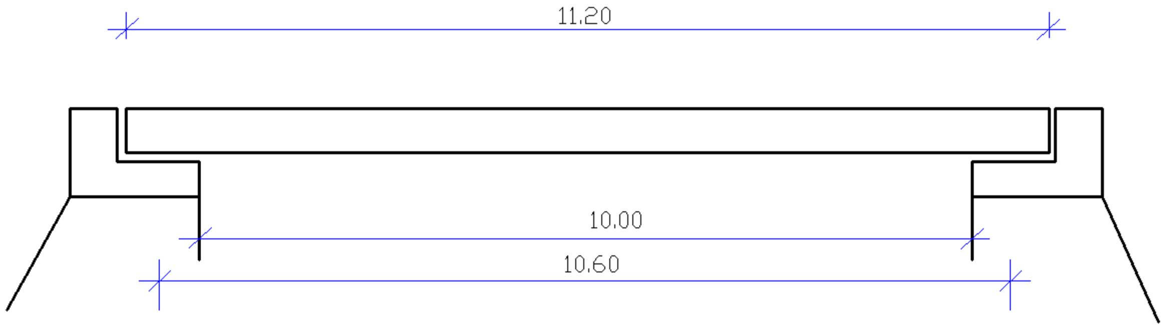
2. Preliminary Dimension



Typical Bridge Cross Section

<u>Overhang:</u>





- **Depth of Girder** (T. 2.5.2.6 3.1.1):

  For a single case: $D = 0.07 \times S,\ S = 10600mm$

  $D = 0.07 \times 10600 = 742mm$

  - $Provide\ depth = 750\text{mm}$

- **Spacing between Girders:**

  Spacing is between 1.80m and 3.60m

  $1.80m < a < 3.60m$

  Cantilever portion, C is between $0.25 \times S\ to\ 0.75 \times S;\ \ S = a$

  $a = spacing\ between\ Girder$

  $2 \times a + 2 \times c = 9.6 \quad let \quad c = 0.5a$

  $a = 3.20m$

  $c = 1.60m$

- **Width of the Web:**

  The minimum thickness is 200mm $/A\ 5.1.4.13.1e/$

  Let's assume there are four bars of $\emptyset 32\ \ with\ clear\ spacing\ 1.5 \times \emptyset$

  $s = 1.5 \times 32 = 48mm$

  $b_w = 40 \times 2 + 2 \times \emptyset 12 + 4 \times \emptyset 32 + 3s = 376mm$

  - $Provide\ Width = 380\text{mm}$

- **Thickness of the Slab,** t_s

  $$t_s = \frac{(S + 3000)}{30}, \quad Where\ \ S = 3.2m = 3200mm$$

  $$t_s = \frac{(3200 + 3000)}{30} = 206.67mm$$

  - $Provide\ Slab\ thickness = 210\text{mm}$

<u>Diaphragms:</u>

Provide diaphragms to the tie girders together so that can resist the load which buckling.

- One at a mid span
- One at each end of the girder

  - Thickness of the diaphragm $= 250mm$

    Depth $= 600mm$

3. Design of Deck Slab:

- Concrete dead weight $= 0.21 \times 24 = 5.04KN/m^2$
- Overhang Slab $= 0.21 \times 24 = 5.04KN/m^2$
- Curb$= 0.25 \times 24 = 6KN/m^2$

$$N_p = \frac{(L+S)}{t_p + S}; \quad Where \;\; L = 11.20m, \quad S = 2.20m$$

$$t_p = 0.3m$$

$$N_p = \frac{(11.20 + 2.20)}{0.3 + 2.20} = 5.36 \approx \mathbf{6.0}$$

$$\text{Posts} = \frac{6 \times (24 \times 0.25 \times 0.3 \times 0.8)}{10.6} = 0.815KN/m$$

$$\text{Railing} = 0.3 \times 0.25 \times 24 = 1.8KN/m$$

Interior Slab Portion:

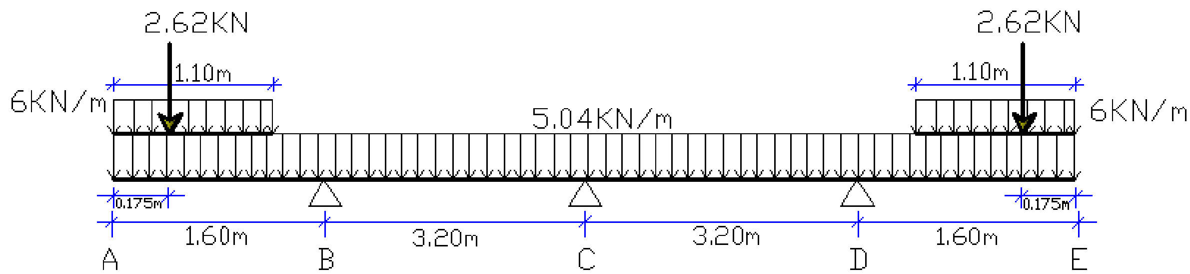
- Dead load moments

An approximate analysis of strips perpendicular to girder will be considered /A 9.6/

The extreme $(+ve)$ moment in any deck panel between girders shall be taken to apply to all $(+ve)$ moment regions.

Similarly, the extreme $(-ve)$ moments over any girder shall be taken to apply to all $(-ve)$ moment regions. /A 4.6.2.1/

Take a meter, strip supported over girder:



Fixed end moments:

$$M_{DE}^F = 6 \times 1.1\left(1.6 - \frac{1.1}{2}\right) + 2.62(1.6 - 0.175) + 5.04 \times 1.6 \times \frac{1.6}{2} = 17.115KN/m$$

$$M_{BA}^F = -17.115KN/m$$

$$M_{CB}^F = \frac{Wl^2}{8} = \frac{5.04 \times 3.2^2}{8} = -6.45KNm$$

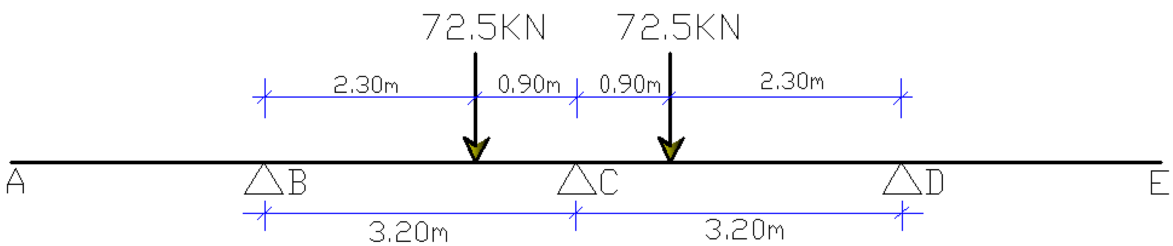
$$M_{CD}^F = \frac{Wl^2}{8} = \frac{5.04 \times 3.2^2}{8} = 6.45KNm$$

Joint	A	B		C		D		E
Member	AB	BA	BC	CB	CD	DC	DE	ED
DF	-	-	1	0.5	0.5	1	-	-
FEM	-	-17.115	-	-6.45	6.45	-	17.115	-
Balance	-	-	17.115	0	0	-17.115	-	-
Carryover	-	-	-	8.56	-8.56	-	-	-
Balance	-	-	-	-	-	-	-	-
Sum	**-**	**-17.115**	**17.115**	**2.11**	**-2.11**	**-17.115**	**17.115**	**-**

- Live load moments:

Maximum $(-ve)$ moment:

Adjacent loading is governing loading position.



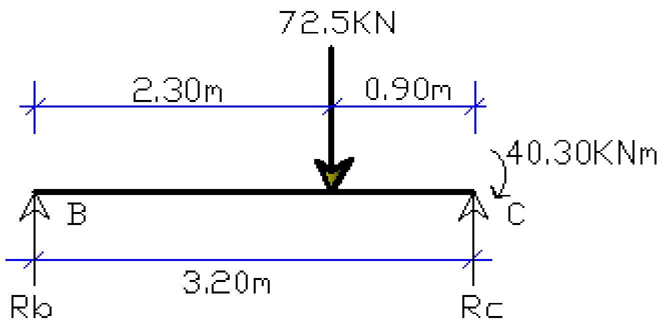
$$M_{AB}^{F} = \frac{p}{l^2}\left(b^2 a + \frac{a^2 b}{2}\right)$$

$$M_{CB}^{F} = \frac{72.5}{3.2^2}\left(2.3^2 \times 0.9 + \frac{0.9^2 \times 2.3}{2}\right) = -40.3KN/m$$

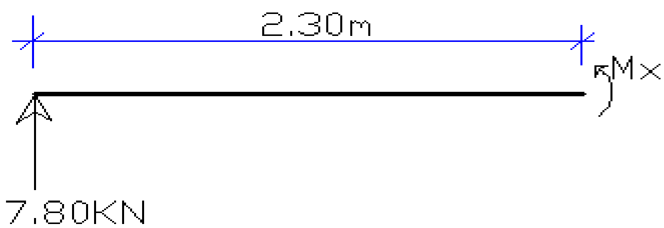
$$M_{CD}^{F} = 40.3KN/m$$

Joint	A	B		C		D		E
Member	AB	BA	BC	CB	CD	DC	DE	ED
DF	-	-	1	0.5	0.5	1	-	-
FEM	-	-	-	-40.30	40.30	-	-	-
Balance	-	-	-	0	0	-	-	-
Sum	-	-	-	-40.30	40.30	-	-	-

Determine max$(+ve)$ moment due to Dead load and Live load.



$$R_b = \frac{72.5 \times 0.9 - 40.3}{3.2} = 7.80KN$$



$$M_{max} = R_B \times 2.3 = 7.8 \times 2.3 = 17.94KNm$$

The width of equivalent transverse strip over which the wheel load acts can be considered distributed in cast-in-place concrete in case of $(+ve)$ moment is $[660 + 0.55 \times S]$.

$$[660 + 0.55 \times S], \qquad [S = a]$$

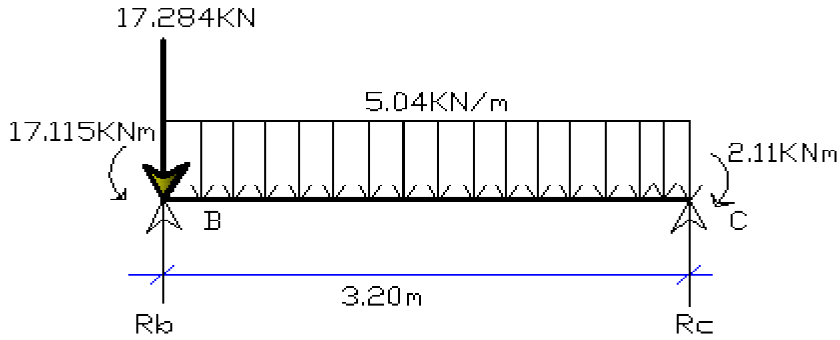
$$[660 + 0.55 \times 3200] = 2420mm = 2.42m$$

Including multiple presence factor, the live load moment: /A 3.6.1.1.2 − 1/

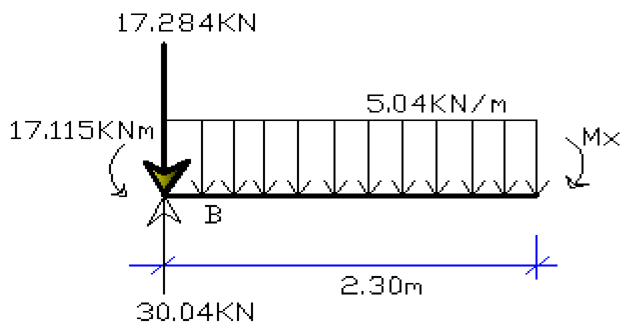
$$M_{max} = \frac{1.2 \times 17.94}{2.42} = 8.90KNm$$

Determine Dead load moment:

$$p = 2.62 + 6 \times 1.1 + 5.04 \times 1.6 = 17.284KN$$



$$\sum M_C = 0, \quad R_B = 30.04KN$$



$$\sum M_x = 0, \quad M_x = -1.11KNm$$

Moment due to dead load $= -1.11KNm/m$ (No need to use Multiple presence factor here)

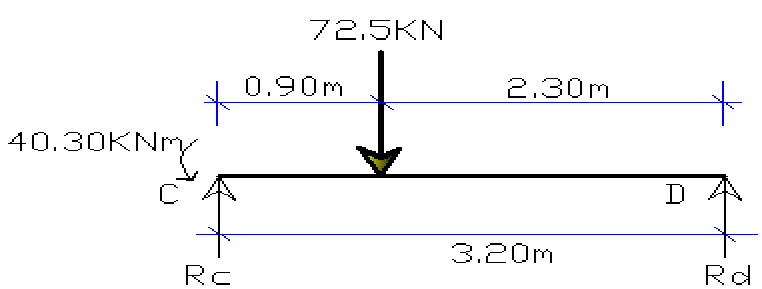
Moment due to live load $= 8.90KNm/m$

<u>Combination for Strength limit state</u>

$$M_{(+ve)} = 1.05[1.25 \times M_{DC} + 1.75 \times M_{LL+IM}]$$

$$M_{(+ve)} = 1.05[1.25 \times -1.11 + 1.75 \times 1.33 \times 8.90] = 20.30KNm/m$$

Determine max$(-ve)$ moment due to Dead load and Live load at face of support or Girder.



$$\sum M_D = 0, \quad R_C = 64.7KN$$

$$\sum M_o = 0, \quad M_o = 17.93KNm/m$$

Dead load:

$$\sum M_x = 0, \quad M_x = 1.11KNm$$

Strip Width:

$$\mathrm{E} = 1220 + 0.25 \times \mathrm{S}, \quad \mathrm{S} = 3200\mathrm{mm}$$

$$\mathrm{E} = 2020\mathrm{mm}$$

$$\mathrm{M}_{face} = \frac{1.2 \times 17.93}{2.02} = 10.65KNm/m$$

Moment due to dead load $= 1.11KNm$

Moment due to live load $= 10.65KNm/m$

Combination for Strength limit state

$$M = 1.05[1.25 \times M_{DC} + 1.75 \times M_{LL+IM}]$$

$$M = 1.05[1.25 \times 1.11 + 1.75 \times 1.33 \times 10.65] = 27.50KNm/m$$

- Design for $(-ve)$ moment:

$$M_{(-ve)} = 27.50KNm/m$$

$Use\ \emptyset 16$

$$d_e = 210 - 60 - \frac{16}{2} = 142mm$$

$Where$:

$$\varphi = 0.9\ ;\ f_y = 300N/mm^2; d = 142mm\ ;\ b = 1000mm$$

$f_c' = 24N/mm^2 \quad M_{(-ve)} = 27.50KNm/m$

$$\rho = \frac{\varphi \times f_c}{f_y}\left(1 - \sqrt{1 - \frac{2M_u}{\varphi \times f_c}}\right) = \frac{0.9 \times 24}{300}\left(1 - \sqrt{1 - \frac{2 \times \frac{27.50 \times 10^6}{bd^2}}{0.9 \times 24}}\right)$$

$$= 0.0048$$

$$\rho_{min} = 0.03\frac{f_c'}{f_y} = 0.03 \times \frac{24}{420} = 0.0017 \ < \rho = 0.0048 \ \ldots\ldots OK!$$

$$A_s = \rho bd = 0.0048 \times 1000 \times 142 = 681.60mm^2$$

$$Spacing\ for\ \emptyset 16 = \frac{\pi \times 8^2 \times 1000}{681.60} = 294.9mm$$

$$Provided \quad \emptyset 16 \ @\ 290mm$$

Checking for over reinforcement:

Maximum reinforcement is limited by ductility requirement, which is given by:

Art 5.7.2.2

$$c/d \leq 0.42, \quad \varphi = 0.9 \quad flexure, \quad \beta = 0.85 \ for\ f_c' \leq 28Mpa$$

$$c/d \leq 0.42$$

$$c = \frac{A_s \times f_y}{(0.85 \times \beta \times f_c' \times b)} = 12.3mm$$

$$\frac{c}{d} \leq 0.42, \quad \frac{12.3}{142} = 0.087 \leq 0.42 \ldots\ldots\ldots Ok!$$

- <u>Design for $(+ve)$ moment</u>:

$$M_{(+ve)} = 20.30KNm/m$$

$$Use\ \emptyset 16$$

$$d_e = 210 - 25 - \frac{16}{2} = 177mm$$

Where:

$\varphi = 0.9\ ;\ f_y = 300N/mm^2; d = 177mm\ ;\ b = 1000mm$

$f_c' = 24N/mm^2 \quad M_{(-ve)} = 20.30KNm/m$

$$\rho = \frac{\varphi \times f_c}{f_y}\left(1 - \sqrt{1 - \frac{2M_u}{\varphi \times f_c}}\right) = \frac{0.9 \times 24}{300}\left(1 - \sqrt{1 - \frac{2 \times \frac{20.30 \times 10^6}{bd^2}}{0.9 \times 24}}\right)$$

$$= 0.003681$$

$$\rho_{min} = 0.03\frac{f_c'}{f_y} = 0.03 \times \frac{24}{420} = 0.0017 \ < \rho = 0.003681 \ldots \ldots OK!$$

$$A_s = \rho bd = 0.003681 \times 1000 \times 142 = 651.54mm^2$$

$$Spacing\ for\ \emptyset 16\ = \frac{\pi \times 8^2 \times 1000}{651.54} = 308.60mm$$

$$Provided \quad \emptyset 16\ @\ 300mm$$

Checking for over reinforcement:

Maximum reinforcement is limited by ductility requirement, which is given by:

Art 5.7.2.2

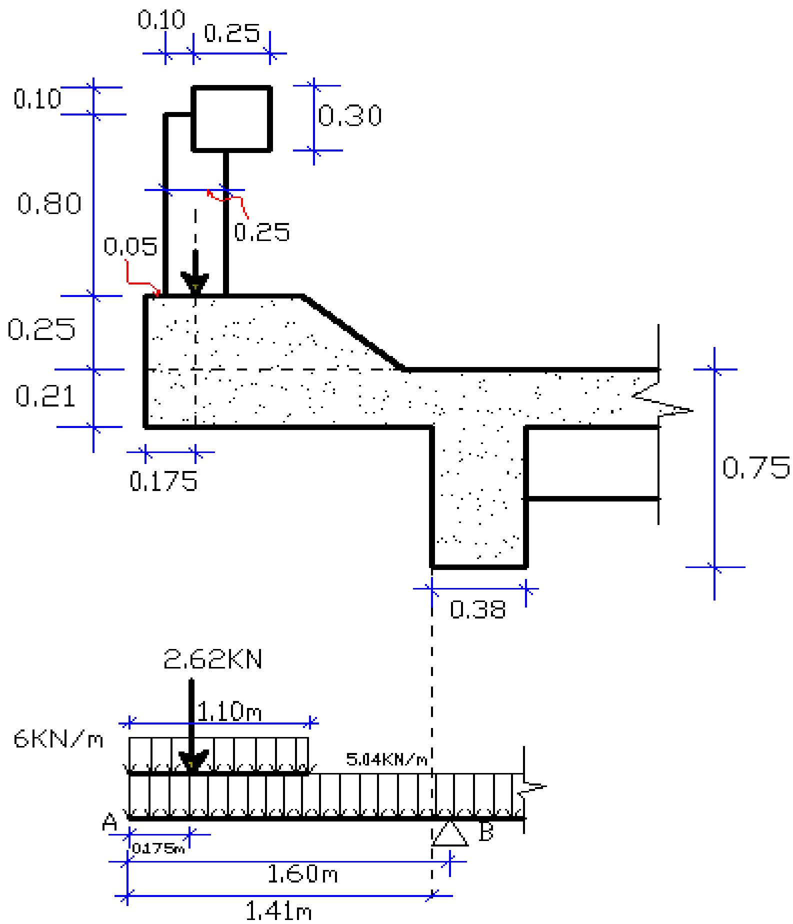
$$c/d \leq 0.42, \quad \varphi = 0.9 \quad flexure, \quad \beta = 0.85\ for\ f_c' \leq 28Mpa$$

$$c/d \leq 0.42$$

$$c = \frac{A_s \times f_y}{(0.85 \times \beta \times f_c' \times b)} = 11.27mm$$

$$\frac{c}{d} \leq 0.42, \quad \frac{11.27}{177} = 0.063 \leq 0.42 \ldots \ldots \ldots Ok!$$

4. Design of Overhanging:



Dead load at the face of the support:

$$M_{A-A}(\text{face of the support}) = 6 \times \frac{(1.1)^2}{2} + 2.62(1.41 - 0.175) + 5.04 \times \frac{(1.41)^2}{2}$$
$$= 11.88KNm/m$$

- Railing Load:

This will act at the top of post and railing:

Railing load shall be applied on an effective length $E = 0.8x + 1.14$. $/A13.4.3.1/$ Where X is the distance from the center of the post to point under investigation (in this case face of support).

The load, $R_1 = 2(1.8KN/m \times 11.20m) = 40.32KN$

This act at a height, $h, above\ the\ point.$

$$h = 0.1 + 0.8 + 0.25 + \frac{0.21}{2} = 1.255m$$

Effective length $E = 0.8x + 1.14$, $x = 1.6 - 0.5 \times 0.38 - 0.175 = 1.235m$

$$E = 0.8x + 1.14 = 0.8(1.235) + 1.14 = 2.128m$$

Therefore, the moment is:

$$M_R = \frac{R_1 \times h}{E} = \frac{40.32 \times 1.255}{2.128} = 23.77KNm/m$$

Load Combination:

Service: $\mathrm{M_s = 1 \times M_{DC} + 1 \times M_{LL}}$

$\mathrm{M_s = 1 \times 11.88 + 1 \times 23.77 = 35.65KNm/m}$

Strength: $\mathrm{M_s = 1.25 \times M_{DC} + 1.75 \times M_{LL}}$

$\mathrm{M_s = 1.25 \times 11.88 + 1.75 \times 23.77 = 56.45KNm/m}$

Provide reinforcement for strength and then check for service

Cover =60mm

$Use\ \emptyset 16$

$$d_e = 210 - 60 - \frac{16}{2} = 142mm$$

$Where$:

$\varphi = 0.9\ ;\ f_y = 300N/mm^2; d = 142mm\ ;\ b = 1000mm$

$f_c' = 24N/mm^2 \quad M = 56.45KNm/m$

$$\rho = \frac{\varphi \times f_c}{f_y}\left(1 - \sqrt{1 - \frac{2M_u}{\varphi \times f_c}}\right) = \frac{0.9 \times 24}{300}\left(1 - \sqrt{1 - \frac{2 \times \frac{56.45 \times 10^6}{bd^2}}{0.9 \times 24}}\right)$$

$$= 0.01003$$

$$\rho_{min} = 0.03\frac{f_c'}{f_y} = 0.03 \times \frac{24}{420} = 0.0017 < \rho = 0.01003 \ \ldots\ldots OK!$$

$$A_s = \rho bd = 0.01003 \times 1000 \times 142 = 1424.33mm^2$$

$$Spacing\ for\ \emptyset 16 = \frac{\pi \times 8^2 \times 1000}{1424.33} = 141.16mm$$

$Provided \quad \emptyset 16\ @\ 140mm$

$$A_s\ Provided = \frac{141.16}{140} \times 1424.33 = 1436.13mm^2$$

Checking for over reinforcement:

Maximum reinforcement is limited by ductility requirement, which is given by:

Art 5.7.2.2

$$c/d \leq 0.42, \quad \varphi = 0.9 \quad flexure, \quad \beta = 0.85 \ for\ f_c' \leq 28Mpa$$

$$c/d \leq 0.42$$

$$c = \frac{A_s \times f_y}{(0.85 \times \beta \times f_c' \times b)} = 24.64mm$$

$$\frac{c}{d} \leq 0.42, \quad \frac{24.64}{142} = 0.173 \leq 0.42 \ldots\ldots\ldots Ok!$$

<u>Distribution Reinforcement:</u>

Secondary reinforcement is placed at the bottom of the slab is longitudinal direction to distribute wheel loads on the bridge to the primary reinforcement in the traverse direction.

Considering the maximum reinforcement, i.e at the overhang

$$A_s \ Provided = 1436.13mm^2$$

%′ age of this A_s

$$\% = min\left(\frac{1750}{\sqrt{L}}, 67\right), \quad Where:$$

$$Girder\ web\ width = 380mm$$

$$L = 3200 - 380 = 2820mm$$

$$\% = min\left(\frac{1750}{\sqrt{2820}}, 67\right) = min(32.95, 67) = 32.95\%$$

Area of reinforcement for secondary reinforcement:

$$= 32.95\%(1436.13)$$

$$= 473.20mm^2$$

$$Spacing\ for\ \emptyset 12 = \frac{\pi \times 6^2 \times 1000}{473.20} = 239mm$$

$$Provided \quad \emptyset 12\ @\ 230mm$$

Shrinkage and temperature reinforcement:

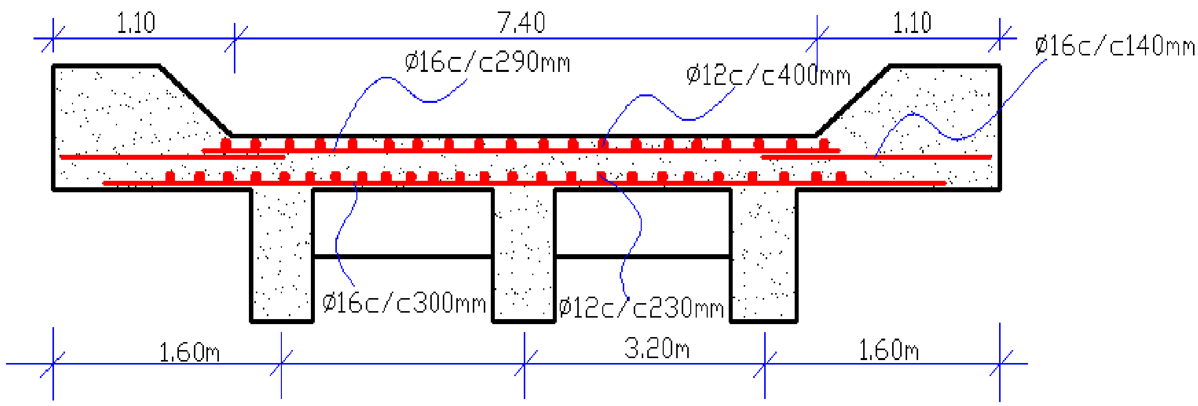
The amount of reinforcement in each direction $0.75A_g/f_y$ Provide half of this reinforcement on top and bottom face.

$$\frac{A_s}{2} = \frac{1}{2}\left(\frac{0.75 \times 210 \times 1000}{300}\right), \quad 210 \rightarrow slab\ thickness$$

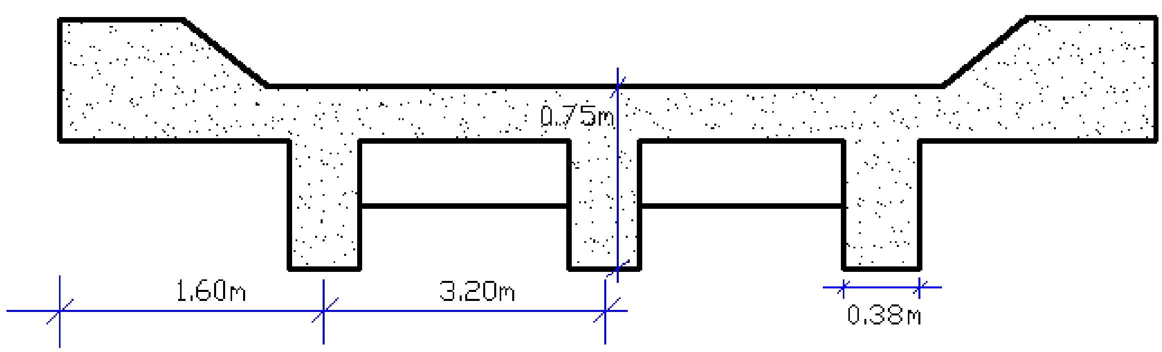
$$= 262.5mm^2$$

$$Spacing\ for\ \emptyset 12 = \frac{\pi \times 6^2 \times 1000}{262.5} = 430.85mm > S_{max} = 400mm$$

$$Provided \quad \emptyset 12\ @\ 400mm$$



5. Design of longitudinal Girders:



Effective flange width $/A\ 4.6.2.6.1/$

$$b_i(interior\ girder) = min\begin{cases} \frac{1}{4}l_{eff} = \frac{1}{4} \times 10600 = 2650mm \\ 12 \times t_s + b_w = 12 \times 210 + 380 = 2900mm \quad \leftrightarrow 2650mm \\ S = 3200mm \end{cases}$$

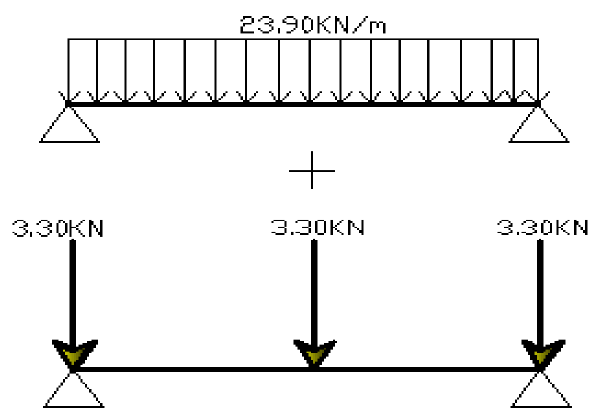
$$b_e(exterior\ girder) = min \begin{cases} \frac{1}{8} l_{eff} = \frac{1}{8} \times 10600 = 1325mm \\ 6 \times t_s + \frac{b_w}{2} = 6 \times 210 + \frac{380}{2} = 1450mm \qquad \leftrightarrow 1325mm \\ Overhang\ length = 1600mm \end{cases}$$

The dead load is computed as follows:

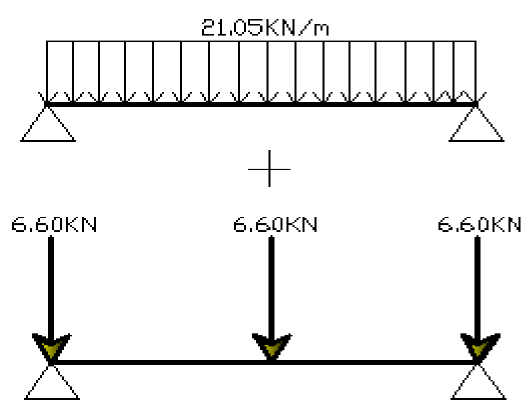
$$\begin{cases} Exterior\ Girder = (2.62 + 6 \times 1.1 + 5.04 \times 1.6) + 1.6 \times 24 \times 0.21 + 0.38 \times (0.75 - 0.21) \times 24 \\ \qquad = 23.9KN \\ Diaphragm\ = (0.6 - 0.21) \times \left(\frac{3.2 - 0.38}{2}\right) \times 24 \times 0.25 = 3.30KN \end{cases}$$

$$\begin{cases} Interior\ Girder = 3.2 \times 0.21 \times 24 + (0.75 - 0.21) \times 0.38 \times 24\ = 21.05KN \\ Diaphragm\ = (0.6 - 0.21) \times (3.2 - 0.38) \times 24 \times 0.25 = 6.60KN \end{cases}$$

Dead load on Exterior Girder:



Dead load on Interior Girder to the longitudinal support.



Factors:

Resistance factor / flexure / (φ):

$$\begin{cases} For\ flexure\ and\ tension = 0.9 \\ For\ shear\ and\ torsion = 0.9 \end{cases}$$

Load Modifiers (A 1. 3.3, A 1.3.4, A 1.2.5)

	Ductility(y_D)	Redundancy(y_R)	Importance(y_I)
Strength	1	1	1.05
Service	1	1	N.A
Fatigue	1	N.A	N.A

Therefore the load modifier's (y_i)

- Strength = $y_D \,.\, y_R \,.\, y_I = 1.05$
- Service = $y_D \,.\, y_R \,.\, y_I = 1$
- Fatigue = $y_D \,.\, y_R \,.\, y_I = 1$

Multiple presence factor $= 1.2$

Impact for fatigue $= 1.15$

Impact for other $= 1.33$

Loading:

- HL - 93 (Spacing 4.3m)

Influence lines will be used for determination of internal action,

- **<u>Moment:</u>**

Distribution factors for Live load: /Table 4.6.2.2.2b-1/

Interior Girder: One lane loaded:

$$M_{i1} = 0.06 + \left(\frac{S}{4267}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{kg}{L \times {t_s}^3}\right)^{0.1},$$

$S = 3200mm,\ \ L = 10600mm,$

$kg\ \ is\ the\ stiffness\ parameter$

$$= 0.06 + \left(\frac{3200}{4267}\right)^{0.4} \left(\frac{3200}{10600}\right)^{0.3} 1 = 0.68$$

Two or more lane loaded:

$$M_{i2} = 0.075 + \left(\frac{S}{2896}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{kg}{L \times {t_s}^3}\right)^{0.1},$$

$S = 3200mm,\ \ L = 10600mm,$
kg is the stiffness parameter

$$= 0.075 + \left(\frac{3200}{2896}\right)^{0.6} \left(\frac{3200}{10600}\right)^{0.2} 1 = 0.91$$

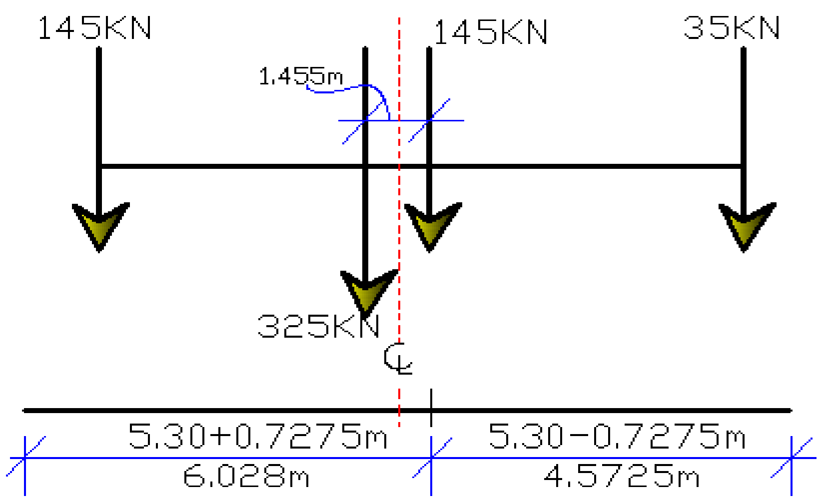
Exterior Girder: Two or more lane loaded:

$$M_{e2} = 1 \times M_{i2} = 1 \times 0.91 = 0.91$$

Influence line for moment:

The absolute maximum moment occurs when axle are placed in such a way that the distance b/n the resultant of the loads and the nearest axle load is equally divided by the center line.

For Truck load:

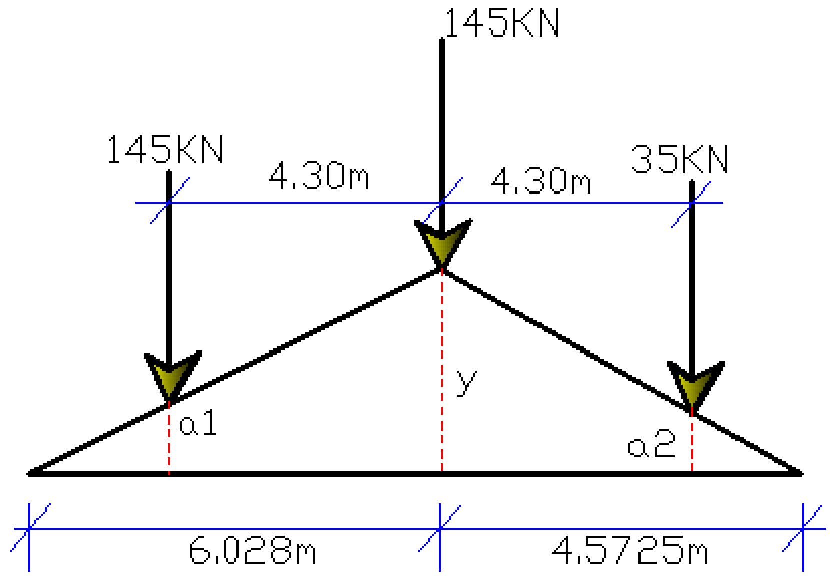


$R = 145 + 145 + 35 = \mathbf{325KN}$

$$x = \frac{145 \times 4.3 + 35 \times 8.6}{325} = 2.845m$$

4.3 – 2.845 = 1.456m

5.3-(1.456/2) = 4.572m



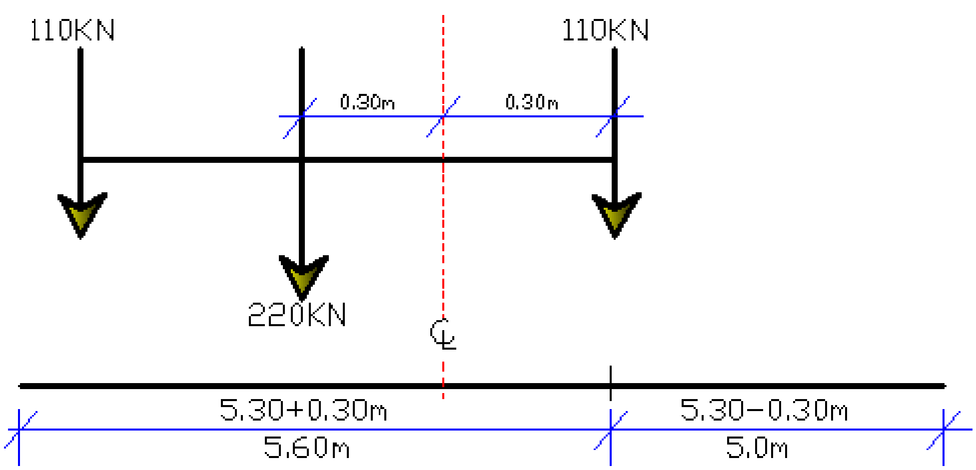
$$y = \frac{(ab)}{L} = \frac{6.028 \times 4.572}{10.6} = 2.6m$$

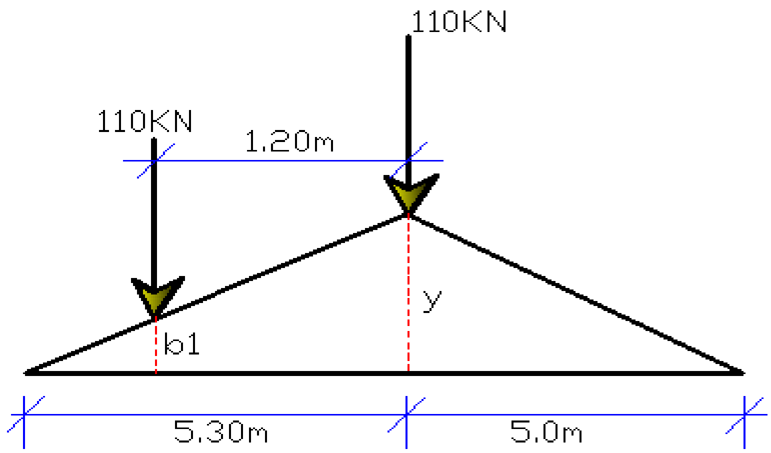
$$a_1 = \frac{2.6 \times 1.7275}{6.028} = 0.745m$$

$$a_2 = \frac{2.6 \times 0.2725}{4.5725} = 0.155m$$

$M = 145(0.745) + 145(2.6) + 35(0.155) = 490.45KNm$

For Tandem load:





$$y = \frac{(ab)}{L} = \frac{5.6 \times 5}{10.6} = 2.64m$$

$$b_1 = \frac{2.64 \times 4.4}{5.6} = 2.074m$$

$$M = 110(2.074) + 110(2.64) = 518.54KNm \qquad \rightarrow Tandem\ load\ governs$$

Shear:

Distribution factor:

Interior Girder: One lane loaded:

$$V_{i1} = 0.36 + \frac{S}{7600}, \qquad S = 3200mm$$

$$V_{i1} = 0.36 + \frac{3200}{7600} = 0.781$$

Multiple lane loaded:

$$V_{i2} = 0.2 + \frac{S}{3600} - \left(\frac{S}{10700}\right)^2, \qquad S = 3200mm$$

$$V_{i2} = 0.2 + \frac{3200}{3600} - \left(\frac{3200}{10700}\right)^2 = 0.999$$

Exterior Girder: One lane loaded (including multiple presence):

$$V_{e1} = 1.2 \times 0.417P, \qquad * 1.2\ multiple\ \text{presence}\ factor$$

$$V_{e1} = 0.5P$$

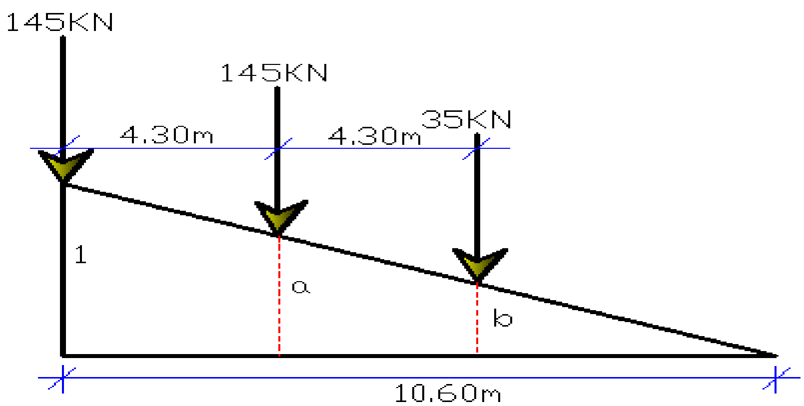
Two or more lane loaded:

$$V_{e2} = V_{i2} \times e \ , \qquad e = 0.6 + \frac{de}{3000}, \qquad de = 250mm$$

$$V_{e2} = 0.999 \times 0.6813 = 0.68$$

Influence line for shear:

For Truck load:

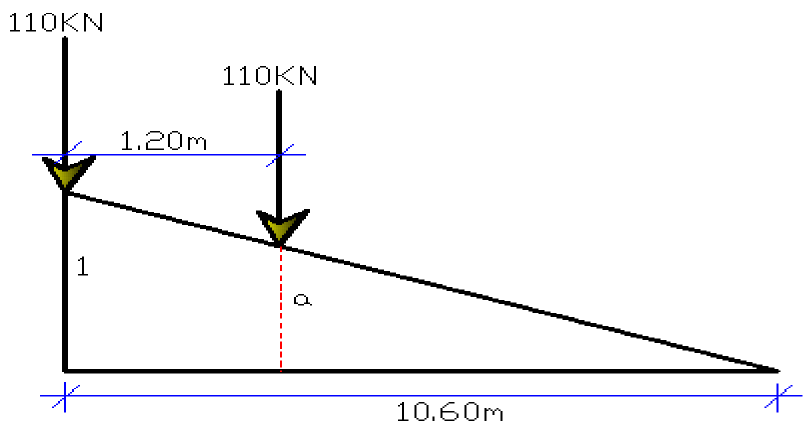


$$a = \frac{6.3 \times 1}{10.6} = 0.594m$$

$$b = \frac{2 \times 1}{10.6} = 0.1887m$$

$$V = 145(1) + 145(0.594) + 35(0.1887) = 237.73KN \rightarrow Truck\ load\ governs$$

For Tandem load:

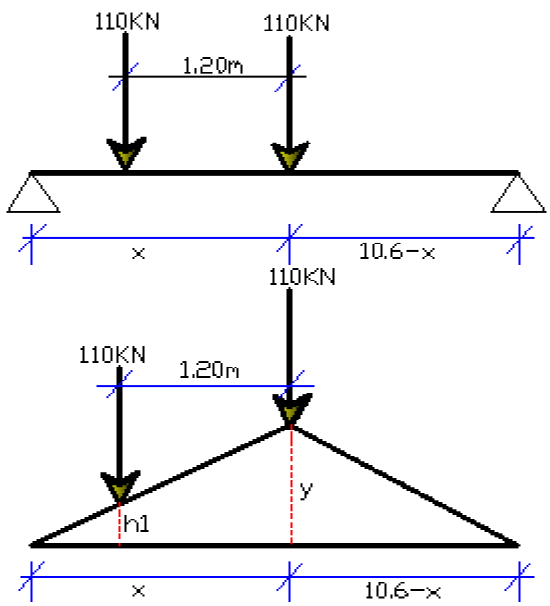


$$a = \frac{9.4 \times 1}{10.6} = 0.887m$$

$V = 110(1) + 110(0.887) = 207.57KN$

		One lane	Multiple lane	
Moment	Exterior Girder	0.69	0.91	$max = 0.91$
	Interior Girder	0.68	0.91	
Shear	Exterior Girder	0.5	0.68	$max = 0.99$
	Interior Girder	0.78	0.99	

Determine (develop governing equations):



$$y = \frac{(ab)}{L} = \frac{x(10.6 - x)}{10.6}$$

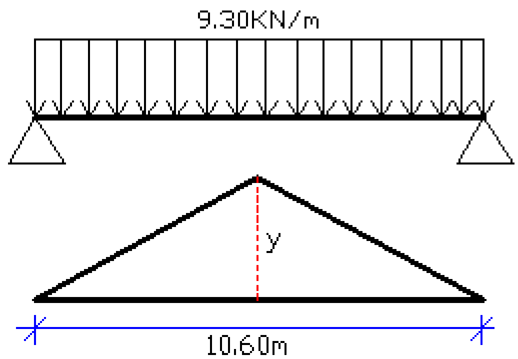
$$h_1 = \frac{(10.6 - x)(x - 1.2)}{10.6} = \frac{-x^2 + 11.8x - 12.72}{10.6}$$

$$M = 110\left(\frac{x(10.6 - x)}{10.6}\right) + 110\left(\frac{-x^2 + 11.8x - 12.72}{10.6}\right)$$

$$= \frac{110}{10.6}(-2x^2 + 22.4x - 12.72)$$

$$= -20.755x^2 + 232.453x - 132$$

For Lane load:



$$V = 9.3 \times 10.6 = 98.58KN$$

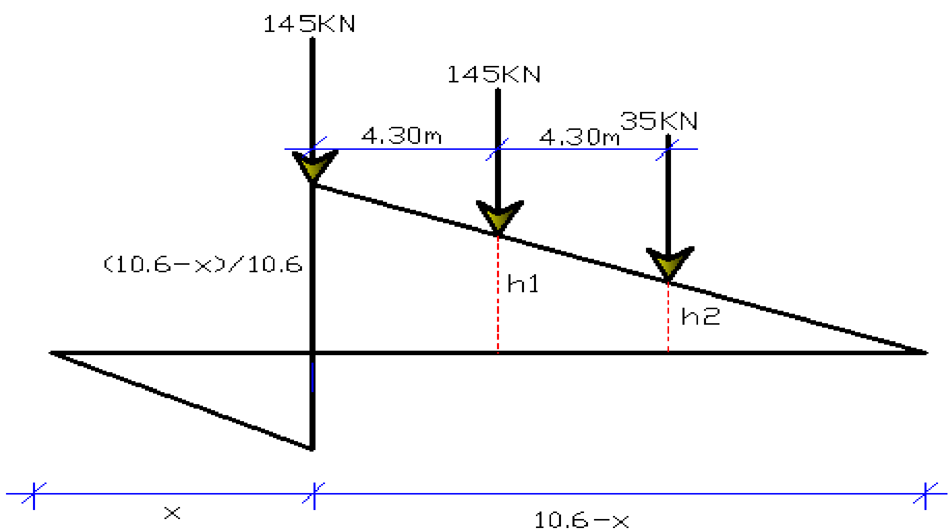
$$Reaction = \frac{98.58}{2} = 49.29KN$$

$$y = \frac{(ab)}{L} = \frac{x(10.6 - x)}{10.6}$$

$$M_{lane(x)} = \frac{1}{2}\left(\frac{x(10.6 - x)}{10.6}\right) \times 10.6 \times 9.3$$

$$M_{lane(x)} = -4.65x^2 + 49.29x$$

For Shear:



$$y = \frac{10.6 - x}{10.6}$$

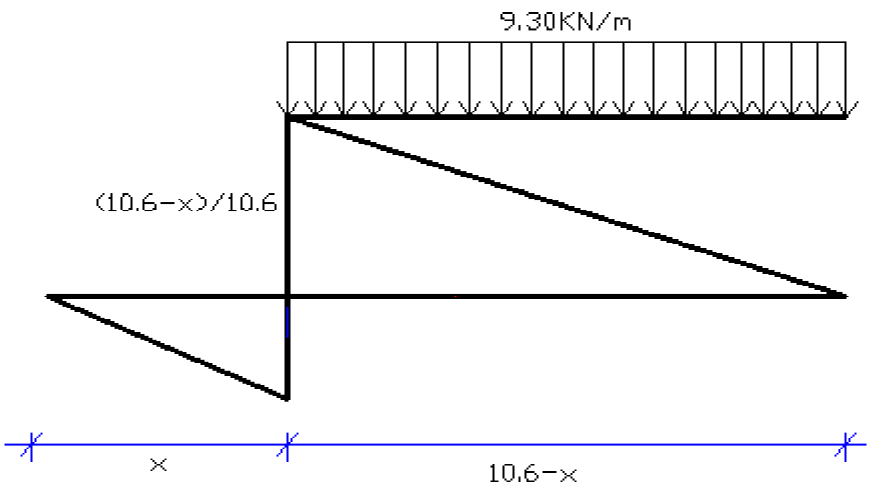
$$h_1 = \frac{6.3 - x}{10.6}$$

$$h_2 = \frac{2 - x}{10.6}$$

$$V_{truck(x)} = 145\left(\frac{10.6 - x}{10.6}\right) + 145\left(\frac{6.3 - x}{10.6}\right) + 35\left(\frac{2 - x}{10.6}\right)$$

$$= \frac{145}{10.6}(10.6 - x + 6.3 - x) + \frac{35}{10.6}(2 - x)$$

$$= -30.662x + 65.424$$



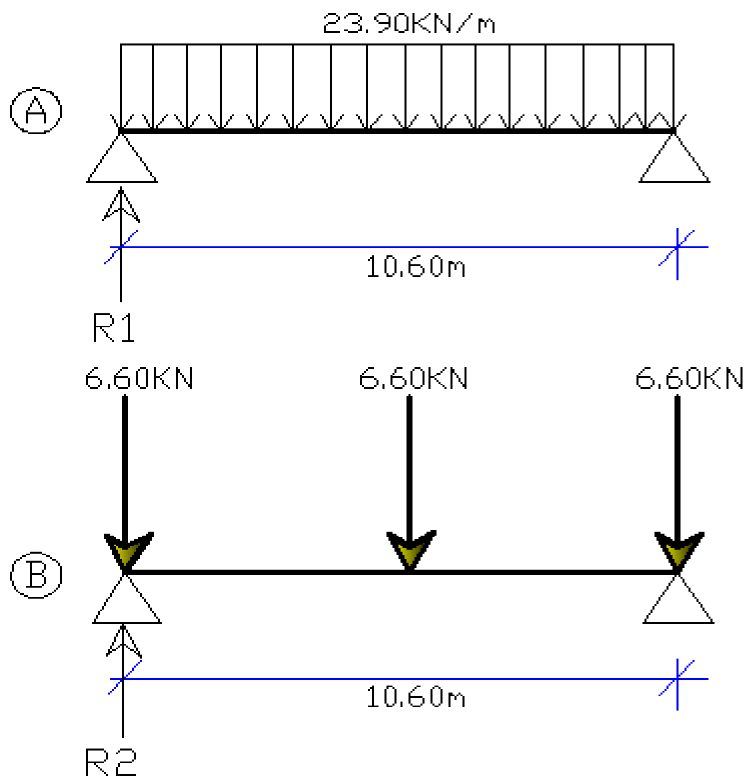
$$V_{lane(x)} = \frac{1}{2}\left(\frac{10.6 - x}{10.6}\right) \times (10.6 - x) \times 9.3$$

$$= 0.4387x^2 - 9.3x + 49.29$$

Distribution factors:

Shear = 0.99, Moment = 0.91

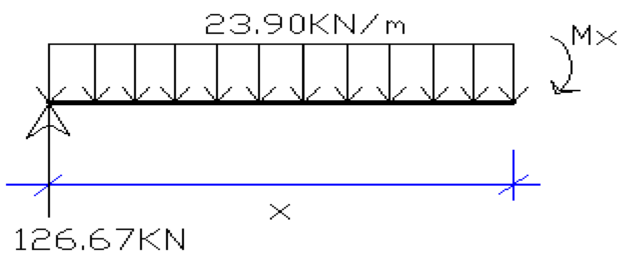
Dead Load:



From: A

$$R_1 = \frac{WL}{2} = \frac{23.9 \times 10.6}{2} = 126.67\text{KN}$$

Moment at any section, from dead load:



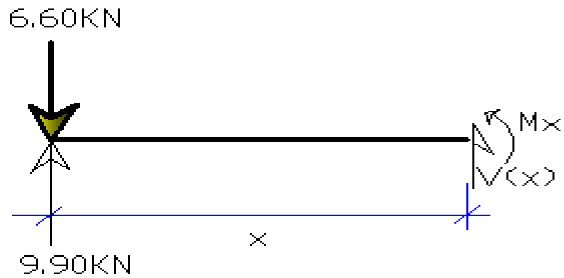
$$M_{(x)} = -11.95x^2 + 126.67x$$

$$V_{(x)} = -23.9x + 126.67$$

From the diaphragm load:

$From$: B

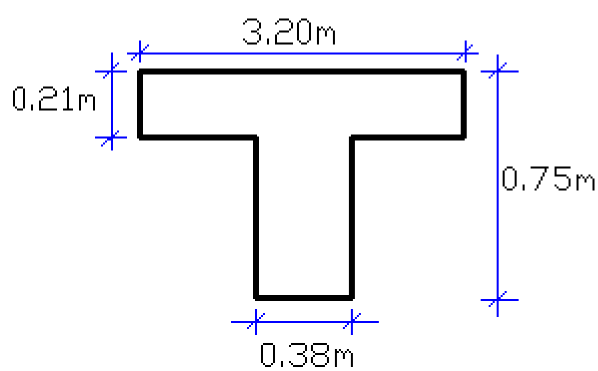
$$R_2 = 6.60 + \frac{6.60}{2} = 9.90\text{KN}$$



$$\begin{cases} V_{(x)} = 9.90 - 6.60 = 3.3\text{KN} \\ M_{(x)} = 3.30x \end{cases}$$

Service limit state:

Since both the interior and exterior girder have similar effective flange width.



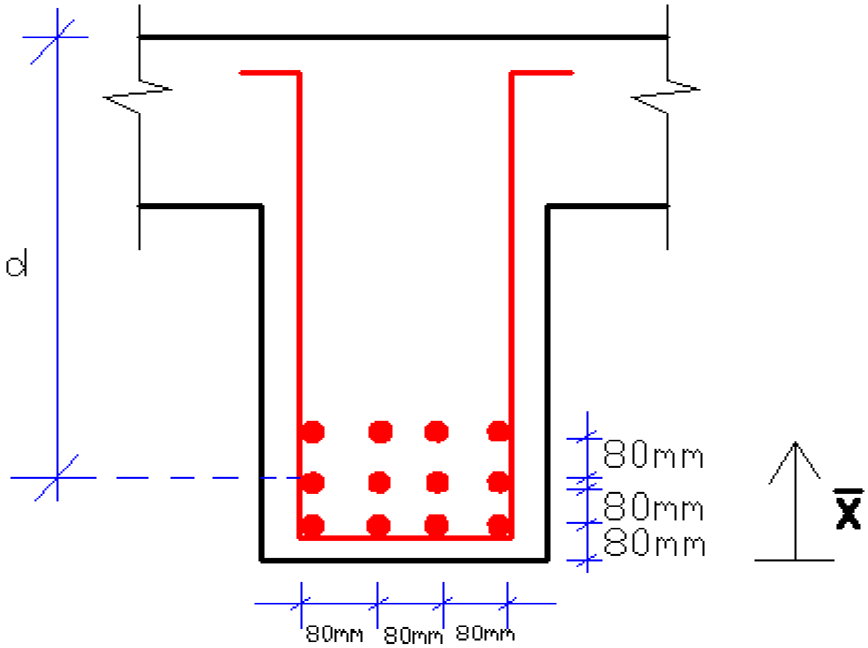
Load Combination:

$$\text{M} = 1[1 \times \text{M}_{\text{DC}} + 1 \times \text{M}_{\text{LL+IM}}]$$
$$\text{M}_{(x)} = 1[1 \times (3.30 - 11.95x^2 + 126.67x) + 0.91 \times 1.33 \times (-20.755x^2 + 232.453x - 132) + 0.91(-4.65x^2 + 49.29x)]$$

$$\text{M}_{(x)} = -41.29x^2 + 452.864x - 156.46, \quad \frac{d\text{M}_{(x)}}{dx} = 0 \ \rightarrow x = 5.5m$$

$$\text{M}_{(5.5)} = 1085.27\text{KNm}$$

Let's try : 12Ø32



$$\overline{x} = \frac{\sum A_i\, x_i}{\sum A_i}$$

$$\overline{x} = \frac{4 \times \pi \times 16^2 \times 80 + 4 \times \pi \times 16^2 \times 160 + 4 \times \pi \times 16^2 \times 240}{12 \times \pi \times 16^2} = 160mm$$

$$\mathrm{d} = (750 - 160)mm = 590mm$$

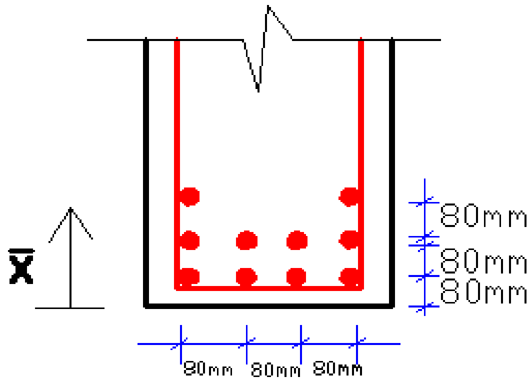
let $\mathrm{j} = 0.875$, $f_y = 420MPa$, $f_s = 0.6 f_y = 0.6 \times 420 = 252MPa$

$$\mathrm{M} = 1085.27\mathrm{KNm}$$

$$A_s = \frac{M}{f_s \times j \times d} = \frac{1085.27 \times 10^6}{252 \times 0.875 \times 590} = 8342.14mm^2$$

$$N_o\ of\ bars = \frac{A_s}{a_s} = \frac{8342.14}{\left(\frac{\pi \times 32^2}{4}\right)} = 10.4$$

$$Use:\ 10bars\ \emptyset 32$$



$$\bar{x} = \frac{\sum A_i\, x_i}{\sum A_i}$$

$$\bar{x} = \frac{4 \times \pi \times 16^2 \times 80 + 4 \times \pi \times 16^2 \times 160 + 2 \times \pi \times 16^2 \times 240}{10 \times \pi \times 16^2} = 144mm$$

$$\mathrm{d} = (750 - 144)mm = 606mm$$

$$A_s = \frac{M}{f_s \times j \times d} = \frac{1085.27 \times 10^6}{252 \times 0.875 \times 606} = 8121.88mm^2$$

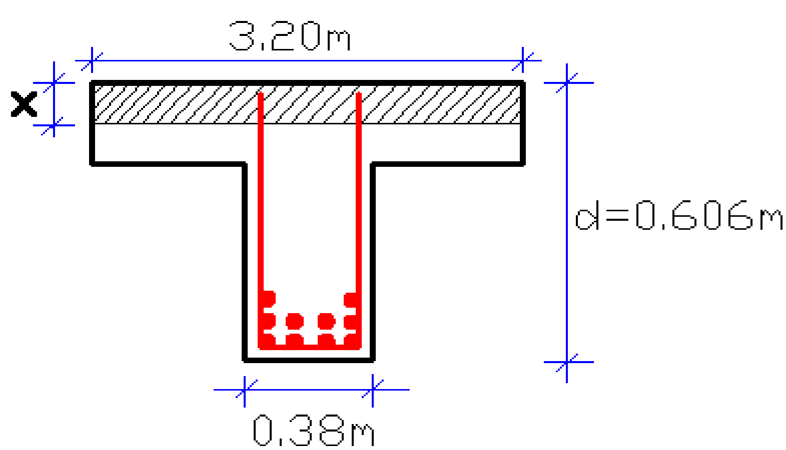
$$N_o\ of\ bars = \frac{A_s}{a_s} = \frac{8121.88}{\left(\frac{\pi \times 32^2}{4}\right)} = 10.1$$

$$Use\colon\ 10bars\ \emptyset 32$$

Checking for Cracking:

Depth of uncracked portion

Assume depth of uncracked portion is within the flange



Using force equilibrium:

$$\frac{1}{2} \times x^2 \times b = n \times A_s \times (d - x),$$

$$\frac{1}{2} \times x^2 \times 3200 = 8 \times (10 \times \pi \times 16^2) \times (d - x), \qquad d = 606mm,$$

$$b = 3200mm, \qquad n = 8$$

$$x = 137.3mm < 210mm(slab\ thickness)$$

Determine the cracked moment of inertia:

$$I_{cr} = \frac{1}{3} \times b \times x^3 + n \times A_s \times (d - x)^2, \qquad x = 137.3mm$$

$$I_{cr} = \frac{1}{3} \times 3200 \times 137.3^3 + 8 \times (10 \times \pi \times 16^2) \times (606 - 137.3)^2$$

$$I_{cr} = 1.69 \times 10^{10} mm^4$$

Stress in the steel:

$$f_s = \frac{n \times M(d - x)}{I_{cr}}, \qquad \mathrm{M} = 1085.27\mathrm{KNm},$$

$$f_s = \frac{8 \times 1085.27 \times 10^6 (606 - 137.3)}{1.69 \times 10^{10}} = 240.79\mathrm{MPa}$$

Determine $f_{sa} = min \begin{cases} 0.6 f_y \\ \frac{Z}{(dA)^{1/3}} \end{cases}$

For moderate exposure (crack width parameter), assumed $Z = 30{,}000 N/mm$

Tensile steel centroid from bottom $\overline{x} = 144mm$

$$A_s = \frac{2 \times 144 \times 380}{10} = 10{,}944 mm^2$$

$$f_{sa} = min \begin{cases} 0.6 f_y = 0.6 \times 420 = 252\mathrm{MPa} \\ \frac{30{,}000}{(10{,}944 \times 50)^{1/3}} = 366.78\mathrm{MPa} \end{cases} \rightarrow 252\mathrm{MPa}$$

Since $f_{sa} > f_s, \leftrightarrow$ $252\mathrm{MPa} > 240.79\mathrm{MPa}$ $\rightarrow$ *the section is safe*!

Deformation:

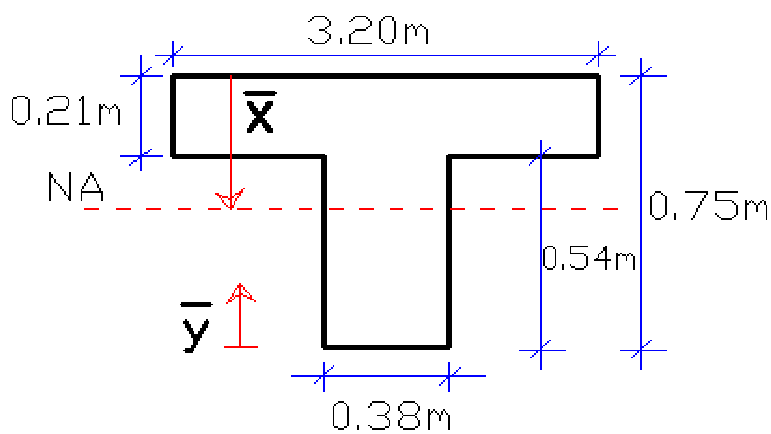
Dead load deflection:

Distribution factor for deflection is the ratio of number of lane and the number of girders.

$$\text{Distribution factor for deflection } = \frac{NL}{NG} = \frac{1}{3} = 0.333,$$

$where$: NL = number of lane,
NG = number of girders

Determine the cracking moment:



Centroid of gross area:

$$\bar{y} = \frac{\sum A_i\, x_i}{\sum A_i} = \frac{540 \times 380 \times 270 + 210 \times 3200 \times 645}{540 \times 380 + 210 \times 3200} = 557.28mm$$

$$\bar{x} = 750 - \bar{y} = 750 - 557.28 = 192.72mm$$

Gross moment of inertia:

$$I_g = \frac{3200 \times 210^3}{12} + 210 \times 3200 \times \left(192.72 - \frac{210}{2}\right)^2 + \frac{380 \times 540^3}{12} + 380 \times 540 \times \left(557.28 - \frac{540}{2}\right)^2$$

$$= 2.9562 \times 10^{10} mm^4$$

Cracking moment, M_{cr}

$$M_{cr} = \frac{f_s \times I_g}{\bar{y}}, \quad f_s = 0.8\left(0.63\sqrt{f_c'}\right) = 0.8\left(0.63\sqrt{24}\right) = 2.49$$

$$M_{cr} = 130.98KNm$$

The moment equation from dead loads:

$$M_{(x)} = M_{dead} + M_{diaphram}$$

$$M_{(x)} = -11.95x^2 + (126.67 + 3.30)x$$

$$= -11.95x^2 + 129.97x$$

Integrating twice:

$$V_{1(x)} = \frac{-11.95x^3}{3} + \frac{129.97x^2}{2} + C_1$$

$$V_{2(x)} = \frac{-11.95x^4}{12} + \frac{129.97x^3}{6} + C_1x + C_2$$

$$V_{(x)} = \frac{1}{EI}\left(V_{2(x)}\right)$$

Deflection at the support = 0

$$V_{(x=0)} = V_{(x=10.6)} = 0, \qquad C_2 = 0$$

$$V_{(x=10.6)} = 0 \rightarrow -1247.85 = C_1$$

$$V_{(x)} = \left(\frac{1}{EI_e}\right)\left[\frac{-11.95x^4}{12} + \frac{129.97x^3}{6} - 1247.85\right]$$

Determine the effective moments of inertia:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \times I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

Determine maximum deflection position so that M_a can be calculate:

Determine maximum deflection:

$$M_{(x)} = -11.95x^2 + 129.97x$$

$$\frac{dM_{(x)}}{dx} = 0, \rightarrow x = 5.44m$$

$$M_a = M_{(x)} = -11.95(5.44)^2 + 129.97(5.44) = 353.40KNm$$

Substituting: $I_{cr} = 1.69 \times 10^{10}mm^4$, $I_g = 2.9562 \times 10^{10}mm^4$,

$$\mathrm{M}_{cr} = 130.98KNm$$

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \times I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} = 1.754 \times 10^{10} mm^4$$

$$E_c \times I_e = 24.768 \times 1.754 \times 10^{10} = 434.43KNmm^2$$

$$\mathrm{V}_{(x)} = \left(\frac{1}{EI_e}\right)\left[\frac{-11.95x^4}{12} + \frac{129.97x^3}{6} - 1247.85\right]$$

$$\mathrm{V}_{(x=5.44)} = \left(\frac{1}{434.43}\right)\left[\frac{-11.95(5.44)^4}{12} + \frac{129.97(5.44)^3}{6} - 1247.85\right]$$

$$\mathrm{V}_{(x=5.44)} = 3.15 \sim 4mm(\text{immediate deflection})$$

Since, I_e is used for deflection computation long term deflection is equal to (immediate deflection) $\times \left(3 - 1.2 \times \frac{A_s'}{A_s}\right)$, A_s' → area of compression reinforcement = 0

$$4 \times \left(3 - 1.2 \times \frac{0}{A_s}\right) = 12mm$$

$$Provide\ \mathbf{15mm}\ Camber$$

<u>Live load deflection:</u>

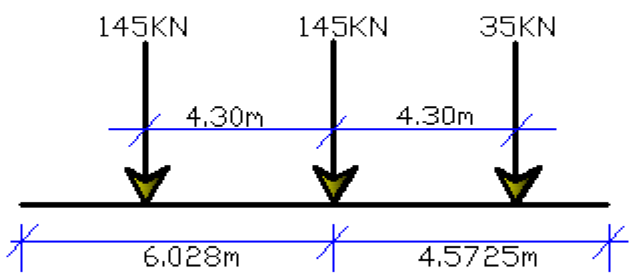
Deflection shall be compared for:

$\rightarrow$ *one design truck*

$\rightarrow$ 25% *design truck plane load*

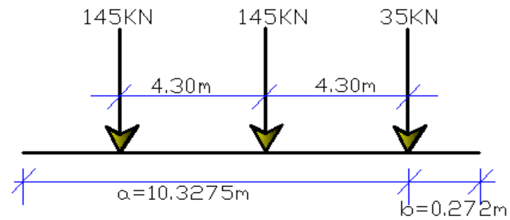
Take the maximum deflection:

Maximum deflection due to truck load occurs when the lane or axle produce maximum moment.



$$\Delta = \frac{Pbx}{6E_cI_eL}(l^2 - b^2 - x^2), x \leq a$$

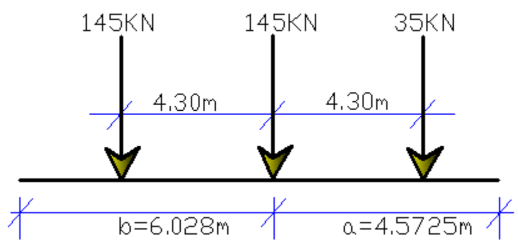
$1^{st} axle; \; P = 35 \times 1.33 \times 1.2 = 55.86KN$



$a = 10.3275m, x = 6.028m, b = 0.272m, L = 10.6m$

$\Delta = 0.2518mm$

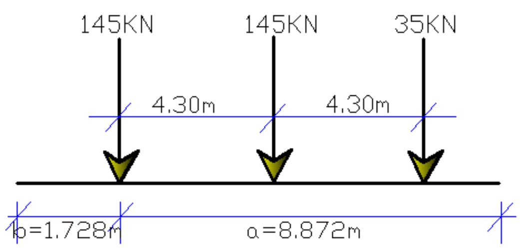
$2^{nd} axle; \; P = 145 \times 1.33 \times 1.2 = 231.42KN$



$a = x = 4.572m, b = 6.028m, L = 10.6m$

$\Delta = 12.724mm$

$3^{rd} axle; \; P = 145 \times 1.33 \times 1.2 = 231.42KN$

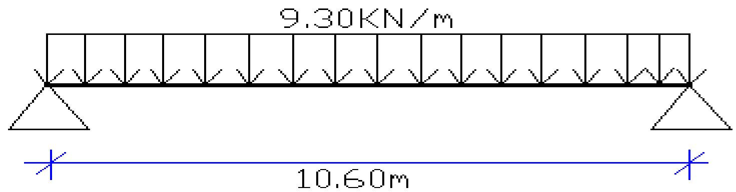


$a = 8.872m, x = 4.572m, b = 1.728m, L = 10.6m$

$\Delta = 5.854mm$

$$Total\ deflection = \frac{1}{3}[0.2518 + 12.724 + 5.854]mm = 6.3mm$$

Due to lane load:



$$\Delta = \frac{5}{384}\frac{Wl^4}{E_c I_e}, \qquad W = 9.3 \times 1.2 = 11.16KN/m$$

$$\Delta = \frac{5}{384} \times \frac{11.16 \times 10.6^4}{434.43} = 4.22mm$$

$$\Delta = M_{max}\begin{cases} Truck\ load\ deflection = 6.3mm \\ Lane\ load\ deflection + 25\%\ truck = 4.22 + \frac{1}{4}(6.3) = 5.795mm \end{cases}$$

$$\Delta = 6.30mm$$

$$Allowable\ deflection: \quad \frac{L}{800} = \frac{10600}{800} = 13.25mm$$

$$6.30mm < 13.25mm \ldots\ldots\ldots\ldots..\boldsymbol{Ok}!$$

<u>Stregth limit state:</u>

Load Combination:

$$M_u = 1.05 \times [1.25 \times M_{DC} + 1.75 \times M_{(LL+IM)}]$$

$$M_u = 1.05 \times [1.25 \times (-11.95x^2 + 129.97x) + 1.75 \times 1.33 \times 0.91 \times (-20.755x^2 + 232.453x - 132) + 1.75 \times 0.91 \times (-4.65x^2 + 49.29x)]$$

$$M_u = -69.62x^2 + 769.96x - 293.56$$

Maximum moment:

$$\frac{dM_{(x)}}{dx} = 0, \rightarrow x = 5.53m$$

$$M_{u(x=5.53)} = 1835.28KNm$$

$$\left[\frac{\varphi \times {f_y}^2}{(1.7 \times f_c' \times b)}\right]{A_s}^2 - (\varphi \times f_y \times d)A_s + M_u = 0$$

$$\varphi = 0.9\ ;\ f_y = \frac{420N}{mm^2}; d = 606mm\ ;\ b = 3200mm;\ f_c' = 24N/mm^2$$

$$A_s = 8385.19mm^2$$

$$N_o\ of\ bars\ \emptyset 32 = \frac{A_s}{a_s} = \frac{8385.19}{\left(\frac{\pi \times 32^2}{4}\right)} = 10.42$$

$$Use:\ 10bars\ \emptyset 32$$

Checking for over reinforcement:

Maximum reinforcement is limited by ductility requirement, which is given by:

Art 5.7.2.2

$$c/d \leq 0.42, \qquad \beta = 0.85\ \ for\ f_c' \leq 28Mpa$$

$$c/d \leq 0.42$$

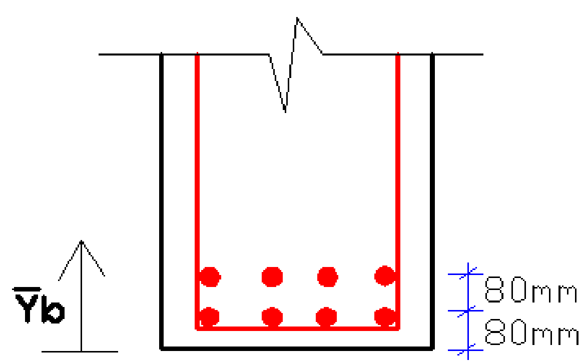
$$c = \frac{A_s \times f_y}{(0.85 \times \beta \times f_c' \times b)} = 60.87mm$$

$$\frac{c}{d} \leq 0.42, \qquad \frac{60.87}{606} = 0.1004 \leq 0.42 \ldots \ldots \ldots Ok!$$

<u>Bar Cutting</u>:

Let’s cut the top two bars where they don’t contribute to the section capacity 8 bars remain.

Determine $d\ and\ \overline{y}_b$



$$\overline{y}_b = \frac{4 \times \pi \times 16^2 \times 80 + 4 \times \pi \times 16^2 \times 160}{8 \times \pi \times 16^2} = 120mm$$

$$d = 750 - \overline{y}_b = 750 - 120 = 630mm$$

$$\mathrm{M}_R = \varphi \times A_s \times f_y \times \left(d - \frac{a}{2}\right), \quad \varphi = 0.9$$

$$a = 0.85c\,, \quad c = \frac{A_s \times f_y}{(0.85 \times \beta \times f_c' \times b)}; \quad A_s = 8 \times \pi \times 16^2 = 6433.98mm^2,$$

$$\beta = 0.85, \qquad b = 3200mm$$

$$c = 48.7mm$$

$$a = 0.85 \times 48.7 = 41.40mm$$

$$\mathrm{M}_R = 1{,}481.84\mathrm{KNm}$$

Solve:

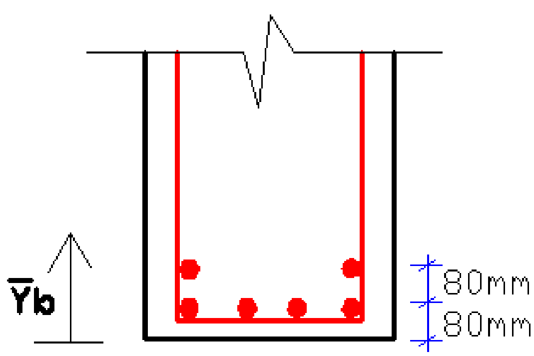
$$\mathrm{M}_R = M_u$$

$$1{,}481.84 = -69.62x^2 + 769.96x - 293.56$$

$$x = 3.27; \quad x = 7.78$$

Cut other two additional bars:

Determine $d \; and \; \overline{y}_b$



$$\overline{y}_b = \frac{4 \times \pi \times 16^2 \times 80 + 2 \times \pi \times 16^2 \times 160}{6 \times \pi \times 16^2} = 106.67mm$$

$$d = 750 - \overline{y}_b = 750 - 106.67 = 643.33mm$$

$$\mathrm{M}_R = \varphi \times A_s \times f_y \times \left(d - \frac{a}{2}\right), \quad \varphi = 0.9$$

$$a = 0.85c\,, \quad c = \frac{A_s \times f_y}{(0.85 \times \beta \times f_c' \times b)}; \quad A_s = 6 \times \pi \times 16^2 = 4825.48mm^2,$$

$$\beta = 0.85, \qquad b = 3200mm$$

$c = 36.52mm$

$a = 0.85 \times 36.52 = 31.05mm$

$\text{M}_R = 1{,}145.14\text{KNm}$

Solve:

$\text{M}_R = M_u$

$1{,}145.14 = -69.62x^2 + 769.96x - 293.56$

$x = 2.38; \;\; x = 8.68$

<u>Development length</u>:

Basic development length:

$$lbd = max\begin{cases} \frac{0.02\times A_b\times f_y}{\sqrt{f_c'}}; & A_b = \pi \times 16^2 \\ 0.06 \times \emptyset \times f_y\,; & \emptyset = 32mm \end{cases}$$

$$= max\begin{cases} 1378.99mm \\ 806.40mm \end{cases} \rightarrow 1380mm\ is\ provided$$

$\rightarrow 6\ bars = continue\ to\ support$

$\rightarrow 2\ bars = (8.68 - 2.38) + 2 \times 1.38 = 9.06m$

$\rightarrow 2\ bars = (7.78 - 3.27) + 2 \times 1.38 = 7.27m$

<u>Shear Reinforcement</u>:

<u>Step - 1</u>:

Determine $V_u\ and\ M_u$ at a distance d_v from face of support:

First determine d_v at support $N_o\ of\ bar\ is\ 6 \;\; \rightarrow \; A_s = 6 \times \pi \times 16^2 = 4825.48mm^2$

$b_v = 380mm$

$b_e = 3200mm$

$$a = 0.85c = \frac{A_s \times f_y}{(\beta \times f_c' \times b)}; \;\; f_y = 420MPa, \;\; f_c' = 24MPa, \;\; \beta = 0.85,$$

$a = 31.04mm$

Effective shear depth, $d = 643.33mm$

$$d_v = d - \frac{a}{2} = 643.33 - \frac{31.04}{2} = 627.81mm$$

$$\text{But, } d_v = max\begin{cases} d - \frac{a}{2} = 627.81mm \\ 0.9d_e = 0.9 \times 643.33 = 579mm \\ 0.72 \times D = 0.72 \times 750 = 540mm \end{cases}$$

$$\rightarrow d_v = 627.81mm$$

$$V_u = 1.05 \times \left[1.25 \times V_{DC} + 1.75 \times V_{(LL+IM)}\right]$$

$$= 1.05 \times [1.25 \times -23.9x + 126.67 + 3.3 + 0.99 \times 1.75 \times 1.33 \times (-30.662x + 65.424 + 0.4387x^2 - 9.3x + 49.29)]$$

$$= 1.05 \times [1.25 \times (-23.9x + 129.97) + 0.99 \times 1.75 \times 1.33 \times (-39.962x + 114.714 + 0.4387x^2)]$$

$$V_u = 1.061x^2 - 128.05x + 448.14$$

$$M_u = -69.62x^2 + 769.96x - 293.56$$

$$V_u(x = d_v = 0.62781) = 368.17KN$$

$$M_u(x = d_v = 0.62781) = 162.39KNm$$

<u>Step - 2</u>:

Calculate the shear stress ratio:

$$v = \frac{V_u}{\varphi \times d_v \times b_v} = \frac{368.17}{0.9 \times 627.81 \times 380} = 1.715N/mm^2$$

$$\frac{v}{f_c'} = \frac{1.715}{24} = 0.0715 < 0.25(width) \rightarrow if\ greater\ the\ larger\ web\ section\ need\ to\ be\ used$$

<u>Step - 3</u>:

Estimate initial value of $\boldsymbol{\theta}$ $and\ \ Calculate\ \boldsymbol{\varepsilon x}$:

$let\ \ \boldsymbol{\theta} = 35^0$

$$\boldsymbol{\varepsilon x} = \frac{\frac{M_u}{d_v} + 0.5 \times V_u \times Cot\theta}{E_s \times A_s}, \quad A_s = 6 \times \pi \times 16^2 = 4825.5mm^2,$$

$$E_s = 200000Mpa$$

$$\boldsymbol{\varepsilon x} = 0.89 \times 10^{-3}$$

Step - 4: Determine concrete capacity:

Take $\beta = 2.27$

$$V_c = 0.083 \times \beta \times d_v \times b_v \times \sqrt{f_c^{'}} \quad , \quad b_v = 380mm$$

$$V_c = 220.2KN$$

Step - 5: Determine shear force by stirrups:

$$(V_c + V_s) \times \varphi = V_u$$

$$V_s = \frac{V_u}{\varphi} - V_c = 188.87KN$$

Step - 6: Provide stirrups:

Use two leg $\emptyset 12$ for stirrup

$$A_v = 2 \times \pi \times 6^2 = 226.19mm^2$$

$$Spacing, \qquad S = \frac{A_v \times f_y \times d_v \times Cot\theta}{V_s} = 350mm$$

Step - 7: Check for spacing:

$$V = 0.1 \times f_c^{'} \times d_v \times b_v = 572.562KN > V_u = 368.17KN \dots\dots \boldsymbol{Ok}!$$

$$Provide: \ S = 350mm$$

Step - 8: Check for longitudinal reinforcement (5.8.3.5-2):

$$A_s \times f_y \geq \frac{M_u}{d_v \times \varphi} + \left(\frac{V_u}{\varphi} - 0.5V_s\right) Cot\theta \ , \quad A_s = 6 \times \pi \times 16^2 = 4825.5mm^2$$

$$A_s \times f_y = 4825.5 \times 420 = 2{,}026{,}710$$

$$\frac{M_u}{d_v \times \varphi} = 287{,}401.18$$

$$\frac{V_u}{\varphi} - 0.5V_s = 314{,}642.78$$

The section satisfies the reinforcement$\dots\dots \boldsymbol{Ok}$!

Appendix 1 C:

C. Sample Steel Reinforced Elastomeric Bearing Design

Design a steel reinforced elastomeric bearing according to AASHTO LRFD code.

1. **Initial design inputs:**
   Dead Load = 76.97KN
   Live Load = 108.71KN
   Horizontal movement of bridge superstructure = Δ_o=25.4mm
   Axis of pad rotation = Transverse
   Rotation construction tolerance = $0.005\ radians$
   Design rotation = $\theta_s = 0.009\ radians$
   Bearing Shape = Rectangular
2. **Bearing Geometry:**
   Bearing width = W =400mm
   Bearing length = L =250mm
   Total unfactored compressive load P_T =185.67KN
   Minimum required area of bearing A_{min} =16838.68mm²
   Bearing area = A = 100000 mm²
3. **Shear deformation (AASHTO LRFD 14.7.5.3.4):**
   Maximum total shear deformation of Elastomer at service limit = $\Delta_s = \Delta_o = 25.4mm$
   $$2\Delta_s = 50.8mm$$
   Elastomeric layer thickness = $h_{ri} = 9.52mm$
   Thickness of top and bottom cover layers(each) = $h_{cover} = 6.35mm$
   $$h_{cover} \le 0.7h_{ri}$$
   Number of interior Elastomeric layers (Excluding exterior layer allowance) = $n_{int} = 5$
   Total Elastomeric thickness = $h_{rt} = 2h_{cover} + n_{int}h_{ri} = 60.3mm$
   $$h_{rt} \ge 2\Delta_s \text{..........................} \boldsymbol{Ok}!$$
4. **Compressive stress (AASHTO LRFD 14.7.5.3.2):**
   Service Average Compressive stress (Total load) $\sigma_S = \frac{P_T}{A} = 1.86\ N/mm^2$
   Service Average Compressive stress (Live load) $\sigma_L = \frac{P_{LL}}{A} = 1.1\ N/mm^2$
   Rectangular shape factor $= S = \frac{LW}{2h_{ri}(L+W)} = 8.08$
   Shear Modulus of Elastomer = G =0.689 N/mm^2
   For bearings subject to shear deformation:
   $$\sigma_S \le 1.66GS$$
   $$1.86 \le 9.24 \text{..........................} \boldsymbol{Ok}!$$
   $$\sigma_L \le 0.66GS$$
   $$1.1 \le 3.67 \text{..........................} \boldsymbol{Ok}!$$

5. Combined Compression and rotation (AASHTO LRFD 14.7.5.3.5):

Rectangular Bearings:

B = length of pad = 250mm

Exterior layer allowance = $n_{ext} = 1.0$

Equivalent number of interior Elastomeric layers $= n = n_{int} + n_{ext} = 6$

Subject to shear deformation: $\sigma_S < 1.87GS\left[1 - 0.2\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2\right]$

$$1.86 \leq 8.27\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\boldsymbol{Ok}!$$

Fixed against shear deformation: $\sigma_S < 2.25GS\left[1 - 0.167\left(\frac{\theta_s}{n}\right)\left(\frac{B}{h_{ri}}\right)^2\right]$

$$1.86 \leq 9.82\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\boldsymbol{Ok}!$$

6. Stability (AASHTO LRFD 14.7.5.3.6):

For free horizontal translation:

$$2A \leq B$$

$$A = \frac{1.92\frac{h_{rt}}{L}}{\sqrt{1+\frac{2L}{W}}} = 0.309$$

$$2A = 0.618$$

$$B = \frac{2.67}{(S+2.0)\left(1+\frac{L}{4.0W}\right)} = 0.229$$

$$2A \leq B$$

$$0.62 \leq 0.235\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\boldsymbol{not\ Ok}!$$

Note- For rectangular bearings where L$> W$, L and W are interchanged.

Bridge Deck fixed against horizontal translation:

$$\sigma_S \leq \frac{\text{GS}}{A-B} = 85.13\ N/mm^2$$

$$1.86 \leq 69.77\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\boldsymbol{Ok}!$$

7. Reinforcement (AASHTO LRFD 14.7.5.3.7):

Service limit state:

Minimum yield strength of steel reinforcement $= f_y = 300\ N/mm^2$

Thickness of steel reinforcement $= h_s$

$$h_{smin} = \frac{3.0h_{max}\sigma_S}{f_y} = \frac{3 \times 9.52 \times 1.86}{300} = 0.203mm$$

Fatigue limit state:

Constant Amplitude Fatigue threshold $= \Delta F_{TH} = 165\ N/mm^2$(Table 6.6.1.2.5-3)

$$h_{smin} = \frac{2.0h_{max}\sigma_L}{\Delta F_{TH}} = \frac{2 \times 9.52 \times 1.1}{165} = 0.127mm$$

Required Minimum reinforcement thickness $= 0.203mm$

Reinforcement thickness $= h_s = 3.175mm$ (AASHTO 14.7.5.3.5)

$h_s \geq h_{smin}$

$3.175 \geq 0.203$…………………….………….. $\boldsymbol{Ok}$!

8. Final Design Summary:

Bearings Width = W = 4000mm

Bearings Length = L = 250mm

Elastomeric layer thickness = $h_{ri} = 9.52mm$

Thickness of top and bottom cover layers(each) = $h_{cover} = 6.35mm$

Number of interior Elastomeric layers (Excluding exterior layer allowance) = $n_{int} = 5$

Total Elastomeric thickness = $h_{rt} = 60.3mm$

Reinforcement thickness = $h_s = 3.175mm$

Total Bearing Thickness = $h_{rt} + h_s(n_{int} + 1) = 80mm$

Appendix 1 D:

D. Sample Structural Design of Reinforced Concrete Cantilever Retaining Wall

- Material properties
  - Concrete = C-30
  - Steel = S_{420}
  - Angle of Internal Friction, $\emptyset = 30^0$
  - $C = 0$
  - $\gamma_s = 20KN/m^3$
  - $\delta = \frac{2}{3}\emptyset = \frac{2}{3} \times 30 = 20°$
  - Dry density = 1.83gm/cc
  - Optimum moisture content (as %) = 6
  - Unit Weight (Bulk density) = 19.398KN/m3
  - Angle of Inclination $\beta = 0^0$
  - Type of Foundation material = Rock
  - Unit Weight of Concrete = 25KN/m^3
  - The foundation material at the bridge site is slightly weathered rock. Addis Ababa Light Rail Transit (AA-LRT) Project E-W Phase (I) Project Geotechnical (Soil) extension Study Report.
    $\boldsymbol{Q_{all} = 600Kpa}$ **(Allowable bearing capacity)**

Distributed Unfactored Loads from Superstructure

a) Dead Load

i. Railway Bridge Dead Load:

Slab = $t \times w \times l \times \gamma_c = 0.22m \times 11.3m \times 11.2m \times 24KN/m^3 = 668.24KN$

Ballast including track ties = $t_b \times w \times l \times \gamma_b = 0.5m \times 10.5m \times 11.2m \times 19KN/m^3 = 1117.2KN$

Track rail inside guard rail and fastening = $l \times \gamma_t \times no.of\ track\ rail = 11.2m \times 3KN/m \times 4 = 134.4KN$

Girder = $t \times w \times l \times \gamma_c \times no.of\ girder = (1 - 0.22)m \times 0.7m \times 11.2m \times\ 24KN/m^3 \times 5 = 733.82KN$

Edge Beam = $t \times w \times l \times \gamma_c \times no.of\ edge\ beam = 0.40m \times 0.70m \times 11.2m \times 24KN/m^3 \times 2$

$= 150.53KN$

Railing = $l \times \gamma_r \times each\ side\ railing = 11.2m \times 0.5KN/m \times 2 = 11.2KN$

Total $D_L = \mathbf{2815.4KN}$

ii. Girder Bridge Dead Load:

Slab and Overhang = $t \times w \times l \times \gamma_c = 0.21m \times 9.6m \times 11.2m \times 24KN/m^3 = 541.9KN$
Girder = $t \times w \times l \times \gamma_c \times no.of\ girder = (0.75 - 0.21)m \times 0.38m \times 11.2m \times\ 24KN/m^3 \times 3$

$= 165.47KN$

Diaphragm = $t \times w \times l \times \gamma_c \times no.of$ diaphragm $= (0.6 - 0.21)m \times 0.25m \times (3.2 - 0.38)m \times\ 24KN/m^3 \times 3 = 39.6KN$

Curb $= area \times l \times \gamma_c \times each\ side\ curb = 0.25m^2 \times 11.2m \times 24KN/m^3 \times 2 = 134.4KN$

Post $= \frac{6\times(24\times0.25\times0.3\times0.8)}{10.6} \times l \times each\ side\ post = \frac{0.815KN}{m} \times 11.2m \times 2$

$= 18.25KN$

Railing = $t \times w \times l \times \gamma_c \times each\ side\ railing = 0.3m \times 0.25m \times 11.2m \times 24KN/m^3 \times 2$

$= 40.32KN$

Total $D_L = \mathbf{939.94KN}$

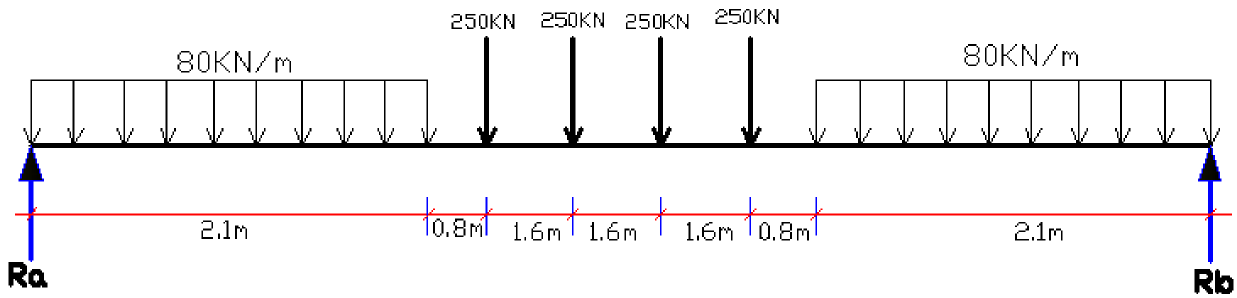
Total Dead Load for both side $= \frac{2815.4\text{KN}+2\times939.94\text{KN}}{2} = \mathbf{2347.64KN}$

Distributed Dead Load $= \frac{\textbf{Dead Load}}{\boldsymbol{Total\ Bridge\ width}} = \frac{\mathbf{2347.64KN}}{\mathbf{30.5}\boldsymbol{m}} = \mathbf{76.97KN/m}$

b) Live Load

i. Railway Bridge Live Load:

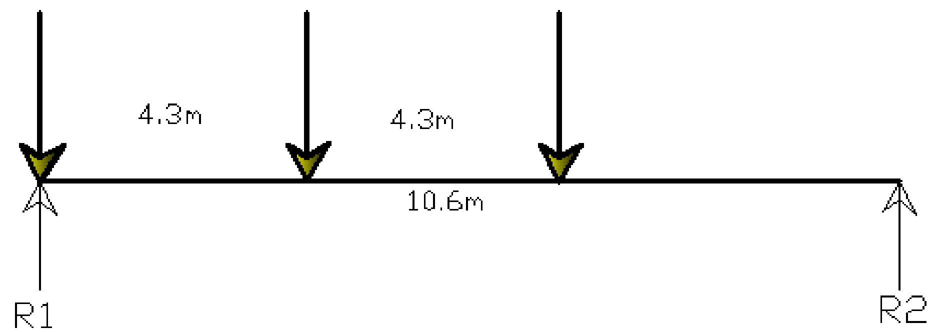
Maximum Live load will be obtained by loading with the nominal type RU loading consisting of four 250 KN concentrated loads spaced at 1.6 meter near the mid span, preceded and followed by uniformly distributed loads of 80 KN/m.



$$R_a = R_b = \frac{1}{2}(80 \times 2.1 \times 2 + 250 \times 4) = 668KN$$

Distributed Live Load Reaction $= \frac{R_a}{Railway\ bridge\ width} = \frac{668KN}{11.3m} = 59.11\ KN/m$

ii. Girder Bridge Live Load:



Max. Live load Reaction will be when one of the wheel loads is laced at one of the supports.

$$\sum MR_1 = 0$$

$$4.3 \times P + 8.6 \times \frac{P}{4} - 10.6 \times R_2 = 0$$

$$R_2 = 0.6085P$$

$$0.6085P + R_1 = 2P + \frac{P}{4}$$

$$R_1 = 1.6415P = 238.02KN$$

Considering two lanes for the critical case

$$R_1 = 2 \times 238.02KN = 476.04KN$$

Distributed Live Load Reaction $= \frac{R_1}{Girder\ bridge\ width} = \frac{476.04KN}{9.6m} = 49.6\ KN/m$

Distributed Live Load = $\mathbf{59.11 + 49.6 = 108.71\ KN/m}$

Lateral Pressure:

The surcharge load from highway loading may be taken as given in Table 3.11.6.4-1. Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic.

Abutment Height	***Equivalent height of soil for vehicular load*** h_{eq}
5ft = 1.52m	4ft = 1.22m
10ft = 3.05m	3ft = 0.91m
≥20ft = 6.10m	2ft = 0.61m

Live load Surcharge:

$\Delta_p = k_a \times \gamma_s \times h_{eq}$

Where:

$\Delta_p =$ constant horizontal earth pressure due to live load surcharge

γ_s = total unit weight of soil

k_a = coefficient of lateral earth pressure

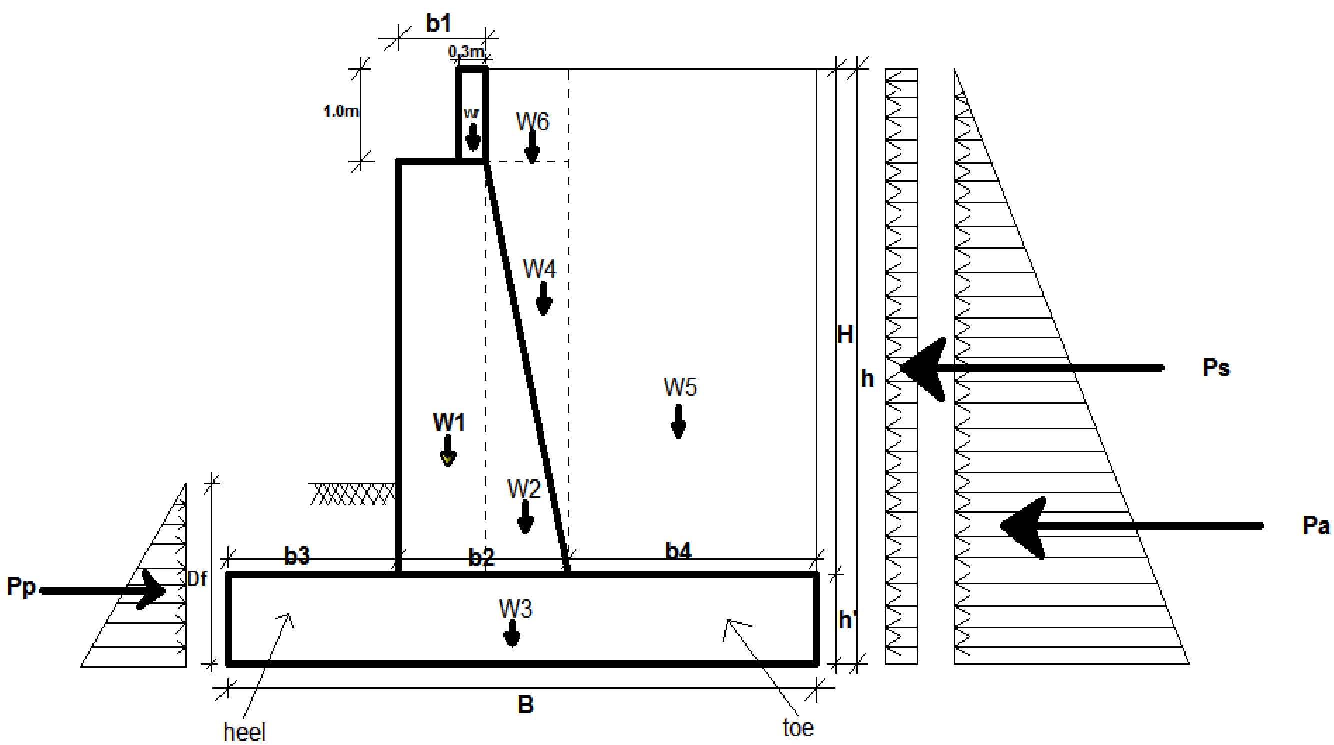
h_{eq} = equivalent height of soil for vehicular load

Equivalent heights of soil, h_{eq} for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1. Linear interpolation shall be used for intermediate wall heights. The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

A table can be prepared for the interpolation between heights of 3.05m and 6.10m.

$\Delta_p = k_a \times \gamma_s \times h_{eq} = 0.33 \times 20 \times 0.6 = 3.96KN/m^2$

Live load Surcharge = $\Delta_p \times Height\ of\ wall = 3.96 \times 6 = \mathbf{23.76KN/m}$



Cantilever retaining wall geometric properties and loading

- Geometric properties of the cantilever retaining wall

$b_1 = 1.0m$	$b_3 = 1.56m$	$D_f = 2m$	$B = 4.67m$	$h = 7.0m$
$b_2 = 1.40m$	$b_4 = 1.71m$	$h' = 1.0m$	$H = 6.0m$	

Analysis and design of retaining walls

- **Vertical loads**

	Vertical load name	Moment arm from toe(m)	Moment about the toe
	$W_1 = 125KN/m$	2.06	256.94
	$W_2 = 25KN/m$	2.69	67.22
	$W_3 = 116.67KN/m$	2.33	272.22
	$W_4 = 20KN/m$	2.82	56.44
	$W_5 = 205.33KN/m$	3.81	782.55
	$W_6 = 8KN/m$	2.76	22.04
	$W_7 = 7.5KN/m$	2.41	18.04
	RDL = 76.97KN/m	2.06	158.22
	RLL= 108.71KN/m	2.06	223.46
	LL Surcharge = 23.76KN/m	3.50	83.16
	Lateral Pressure = 161.7KN/m	2.33	377.30
SUM	**878.64**KN/m		**2,317.6**KNm

- **Lateral loads**

Rankines Method

$k_a = \tan(45 - \emptyset/2)^2 = \tan(45 - 30/2)^2 = 0.33$
$k_p = \tan(45 + \emptyset/2)^2 = \tan(45 + 30/2)^2 = 3$

$p_a = 0.5k_a\gamma_s h^2$
$= 0.5 \times 0.33 \times 20 \times 7^2 = 161.70KN/m \; with \; a \; moment \; arm \; of \; x = 7/3 = 2.33m$
$= 2.33 \times 161.70$ = ***377.3KNm***

$$p_p = 0.5k_p\gamma_s h^2 = 0.5 \times 3 \times 20 \times 2^2 = 120KN/m\ with\ a\ moment\ arm\ of\ x = 0.67m$$

= 0.67x120 = ***80KNm***

A.Stability analysis

1. **Stability against Overturning:**

$$M_{over\ turning} = 377.30$$
$$M_{resisting} = 2{,}317.60$$

$$FS = \frac{M_{resisting}}{M_{over\ turning}} = \frac{2{,}317.60}{377.30} = 6.14 > 1.5 \ldots\ldots\ldots SAFE$$

NB: the passive pressure has been ignored

2. **Stability against Sliding**

$$F_r = f_f + p_p \ldots\ldots\ldots\ldots\ldots ignore\ passive\ pressure$$

$$F_r = f_f = \mu N + C_a B = N\tan\delta + C_a\beta$$

Where: $\delta = \frac{2}{3}\emptyset = \frac{2}{3} \times 30 = 20$

$$C_a = 0.6c = 0.6 \times 0 = 0$$

$$N = \sum W_i = 878.64KN/m$$

$$F_r = 878.64\tan 20 = 319.8KN/m$$

$$Fa = P_a - p_p = 161.7 - 120 = 41.70KN/m$$

$$F_s = \frac{F_r}{F_a} = \frac{319.8}{41.70} = 7.67 > 1.5 \ldots\ldots\ldots SAFE$$

Note:- **If FS < Fs allowable consider the effect of Pp(Passive resistance).**

3. **Stability against Bearing**

$$\sum M = (2{,}317.60 + 80) - (377.30) = 2{,}020.3KNm/m\ and$$

$$\sum V = 878.64KN/m$$

$$X = \frac{2{,}020.3}{878.64} = 2.30$$

$$e_{with\ respect\ to\ toe} = 0.5B - X = 0.5(4.67) - 2.30 = 0.03$$

$$\delta_{max} = \frac{p}{b}\left(1 + \frac{6e_x}{b}\right) = \frac{878.64}{4.67}\left(1 + \frac{6 \times 0.03}{4.67}\right) = \mathbf{196.51} < Q_{all} = 600kpa \ldots\ldots\ldots\ldots.SAFE$$

$$\delta_{min} = \frac{p}{b}\left(1 - \frac{6e_x}{b}\right) = \frac{878.64}{4.67}\left(1 - \frac{6 \times 0.03}{4.67}\right) = \mathbf{180.05} > 0 \ldots\ldots\ldots\ldots\ldots.SAFE$$

B. Structural design of the stem of the retaining walls

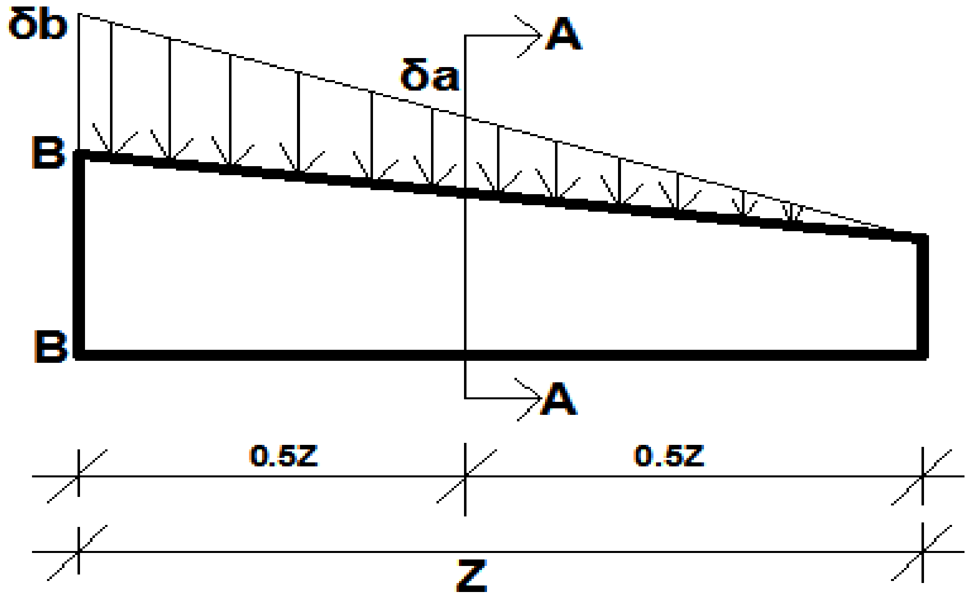
Design Method: working stress design method (WSD)

- The design is based on one meter strip of retaining wall
- Loading of different parts of the cantilever retaining walls

Design Constants:

- $f_{yk} = 420$, $f_{yd} = {f_{yk}}/{1.15} = 365.21$
- $C - 30$, $f_{ck} = 0.8 \times 30 = 24$, $f_c = 0.425 f_{ck} = 0.425 \times 30 = 12.75$
- $f_{ctk} = 0.21 {f_{ck}}^{2/3} = 1.747$, $f_{ctd} = {f_{ctk}}/{1.5} = 1.1647$
- $f_s = 0.52 \times f_{yk} = 0.52 \times 420 = 218.4Mpa$
- $E_c = 200000Kpa$
- $E_c = 5700\sqrt{f_{ck}} = 5700\sqrt{24} = 27924.18Kpa$
- $n = \frac{E_s}{E_c} = \frac{200000}{27924.18} = 7.16$
- $K = \frac{n f_c}{(f_s + n f_c)} = \frac{7.16 \times 12.75}{(218.4 + 7.16 \times 12.75)} = 0.295$
- $j = 1 - \frac{K}{3} = 1 - \frac{0.295}{3} = 1 - 0.098 = 0.9$
- $R = \frac{K \times j \times f_c}{2} = \frac{0.295 \times 0.9 \times 12.75}{2} = 1.692$
- $\gamma = \frac{f_s}{f_c} = \frac{218.4}{12.75} = 17.13$
- $P_b = \frac{n}{(2\gamma(n+\gamma))} = \frac{7.16}{(2 \times 17.13(7.16 + 17.13))} = 0.0086$
- $\rho_{max} = 0.75 P_b = 0.75 \times 0.0086 = 0.00645$
- $\rho_{min} = 0.0015$

1. Design of Stem:



a) At Section A-A

$\delta_a = \mathrm{q(a-a\,)} = 0.5 \times \mathrm{Z} \times \gamma \times k_a$ = 0.5*5*20*0.33 **= 16.50**

$\delta_b = \mathrm{q(b-b\,)}$ =5*20*0.33 **= 33.00**

M_{a-a} = (0.5***16.50***5/2)*5/4 = **25.78KNm**

V_{b-b} = 0.5*16.50*5/2 = **20.63KN**

Check the effective depth for flexure.

$$D_{req.} \geq \sqrt{\frac{M_{a-a}}{R \times b}} \times 1000 = \sqrt{\frac{25.78}{1.692 \times 1000}} \times 1000 = 123.44mm$$

Use: Ø20mm, cover =80mm

$$D_{req.} = d + 0.5\emptyset + cover = 123.44 + 0.5 \times 20 + 80 = 213.44mm$$

$$D_{provided} = 1.2m = 1200mm$$

$$D_{avail.} = D_{provided} - 0.5\emptyset - cover = 1200 - 0.5 \times 20 - 80 = \mathbf{1110mm}$$

$$D_{req.} = 213.44mm < D_{provided} = 1200mm \ldots\ldots\ldots\ldots Ok!$$

Reinforcement Calculation:

$$A_{Smin} = \rho_{min} \times b \times d = 0.0015 \times 1000 \times 1{,}110 = 1665mm^2/m$$

$$A_S = \frac{M}{f_s \times j \times d} = \frac{25.78}{218.4 \times 0.9 \times 1.11} = 118.16 mm^2/m$$

$$A_S\ used = 1665 mm^2/m$$

$$Spacing:\ S = \frac{A_b}{A_S} \times 1000 = \frac{\frac{\pi \times 20^2}{4}}{1665} \times 1000 = 188.7mm$$

$$\boldsymbol{Use\ \ \emptyset 20\ c/c\ 180mm}$$

$Check\ the\ effective\ depth\ for\ Shear:$

$$Acting\ shear\ stress = \frac{V_{b-b}}{bd} = \frac{20.63}{1000 \times 1110} = 0.0186Kpa$$

$$Resisting\ shear\ stress = 0.0594\sqrt{24} = 0.29Kpa$$

$$Acting\ Shear < Resisting\ shear\ stress$$

b) <u>At Section B-B</u>

M_{a-a} = (0.5***33***5/2)*5/4 = **51.56KNm**

V_{b-b} = (0.5***33***5/2) = **41.25KN**

$D_{provided} = 1400mm$

Check the effective depth for flexure:

$$D_{req.} \geq \sqrt{\frac{M_{a-a}}{R \times b}} \times 1000 = \sqrt{\frac{51.56}{1.692 \times 1000}} \times 1000 = 174.57mm$$

Use Ø30mm

$$D_{req.} = d + 0.5\emptyset + cover = 174.57 + 0.5 \times 30 + 80 = 269.57mm$$

$$D_{avail.} = D_{provided} - 0.5\emptyset - cover = 1400 - 0.5 \times 30 - 80 = \mathbf{1305mm}$$

$$D_{req.} = 269.57mm < D_{provided} = 1400mm \ldots\ldots\ldots\ldots Ok!$$

<u>Reinforcement Calculation:</u>

$$A_{Smin} = \rho_{min} \times b \times d = 0.0015 \times 1000 \times 1305 = 1957.50 mm^2/m$$

$$A_S = \frac{M}{f_s \times j \times d} = \frac{51.56}{218.4 \times 0.9 \times 1.305} = 201 mm^2/m$$

$$A_S\ used = 1957.50 mm^2/m$$

$$Spacing:\ S = \frac{A_b}{A_S} \times 1000 = \frac{\frac{\pi \times 30^2}{4}}{1957.50} \times 1000 = 361mm$$

$\boldsymbol{Use\ \ \emptyset 30\ c/c\ 360mm}$

$Check\ the\ effective\ depth\ for\ Shear:$

$$Acting\ shear\ stress = \frac{V_{b-b}}{bd} = \frac{41.25}{1000 \times 1305} = 0.0316Kpa$$

$$Resisting\ shear\ stress = 0.0594\sqrt{24} = 0.29Kpa$$

$$Acting\ Shear < Resisting\ shear\ stress$$

c) At front face:

$Minimum\ reinforecement\ shall\ be\ provided\ both\ in\ the\ vertical\ \&\ horizontal\ direction.$

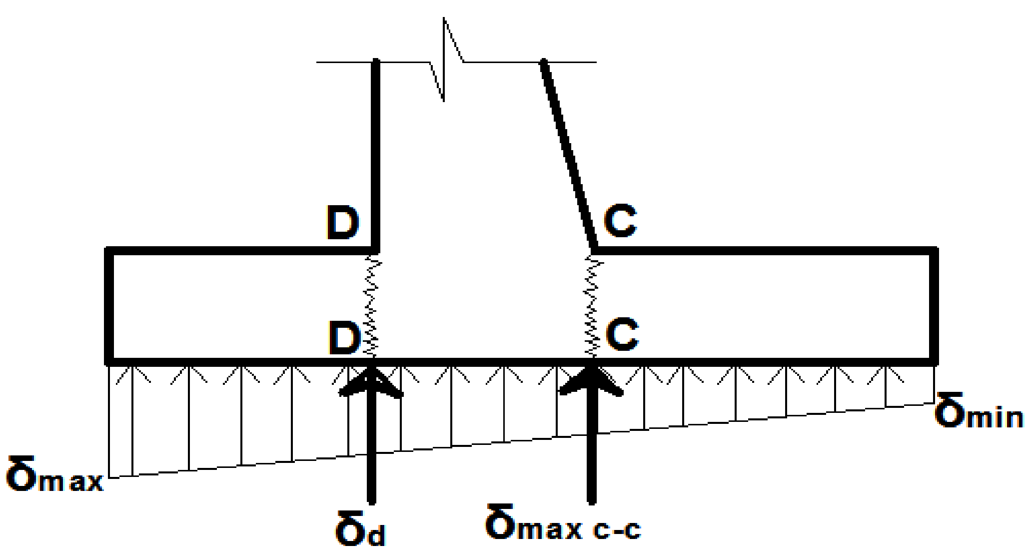
$Use\ \emptyset 20mm\ and\ concrete\ cover = 80mm.$

$$A_{Smin} = 1957.50mm^2/m$$

$$Spacing:\ S = \frac{A_b}{A_S} \times 1000 = \frac{\frac{\pi \times 20^2}{4}}{1957.50} \times 1000 = 160.5mm$$

Use: Ø20 c/c 160mm

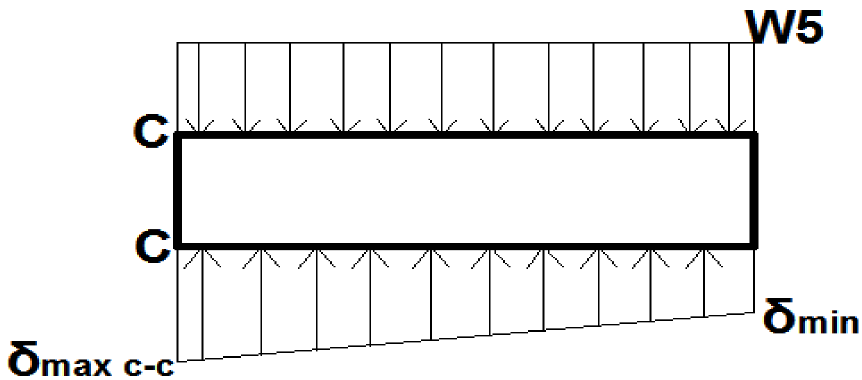
2. Design of Toe:



$$\delta_{max} = \frac{p}{b}\left(1 + \frac{6e_x}{b}\right) = \frac{878.64}{4.67}\left(1 + \frac{6 \times 0.03}{4.67}\right) = 196.51Kpa$$

$$\delta_d = \frac{3.11 \times (196.51 - 180.05)}{4.67} + 180.05 = 191.01Kpa$$

a) <u>At Section C-C</u>



$$W5 = \frac{205.33}{1.71} = 120.07kN/m$$

$$\delta_{max\ c-c} = \frac{1.71 \times (196.51 - 180.05)}{4.67} + 180.05 = 186.08Kpa$$

$$\delta_{min} = \frac{p}{b}\left(1 - \frac{6e_x}{b}\right) = \frac{878.64}{4.67}\left(1 - \frac{6 \times 0.03}{4.67}\right) = 180.05Kpa$$

<u>At Section C-C</u>

$$M_{c-c} = 1.71 \times 180.05 \times \frac{1.71}{2} + 0.5 \times \frac{1.71 \times (196.51 - 180.05)}{4.67} \times 1.71 \times \frac{1.71}{3}$$
$$= 266.18KNm$$

$$= W5 \times 1.71 \times \frac{1.71}{2} = 120.07 \times 1.71 \times \frac{1.71}{2} = 175.55KNm$$

$$= 175.55 - 266.18 = \mathbf{-90.63KNm}$$

$$V_{c-c} = 1.71 \times 180.05 + 0.5 \times \frac{1.71 \times (196.51 - 180.05)}{4.67} \times 1.71$$

$$= 313.04KN$$

$$= W5 \times 1.71 = 120.07 \times 1.71 = 205.32KN$$

$$= 205.32 - 313.04 = \mathbf{-107.72KNm}$$

$$D_{provided} = 1000mm$$

Check the effective depth for flexure:

$$D_{req.} \geq \sqrt{\frac{M_{c-c}}{R \times b}} \times 1000 = \sqrt{\frac{90.63}{1.692 \times 1000}} \times 1000 = 231.44mm$$

Use Ø24mm, Cover = 80mm

$$D_{req.} = d + 0.5\emptyset + cover = 231.44 + 0.5 \times 24 + 80 = 323.44mm$$

$$D_{avail.} = D_{provided} - 0.5\emptyset - cover = 1000 - 0.5 \times 24 - 80 = \mathbf{908mm}$$

$$D_{req.} = 323.44mm < D_{provided} = 1000mm \ldots\ldots\ldots\ldots Ok!$$

Reinforcement Calculation:

$$A_{Smin} = \rho_{min} \times b \times d = 0.0015 \times 1000 \times 908 = 1362mm^2/m$$

$$A_S = \frac{M_{c-c}}{f_s \times j \times d} = \frac{90.63}{218.4 \times 0.9 \times 0.908} = 507.80mm^2/m$$

$$A_S\ used = 1362mm^2/m$$

$$Spacing:\ S = \frac{A_b}{A_S} \times 1000 = \frac{\frac{\pi \times 24^2}{4}}{1362} \times 1000 = 332mm$$

$$\boldsymbol{Use\ \ \emptyset 24\ c/c\ 330mm}$$

$Check\ the\ effective\ depth\ for\ Shear:$

$$Acting\ shear\ stress = \frac{V_{c-c}}{bd} = \frac{107.72}{1000 \times 908} = 0.118Kpa$$

$$Resisting\ shear\ stress = 0.0594\sqrt{24} = 0.29Kpa$$

$Acting\ Shear < Resisting\ shear\ stress$

$Minimum\ reinforecement\ shall\ be\ provided\ both\ in\ the\ vertical\ \&\ horizontal\ direction.$

$Use\ \emptyset 20mm\ and\ concrete\ cover = 80mm.$

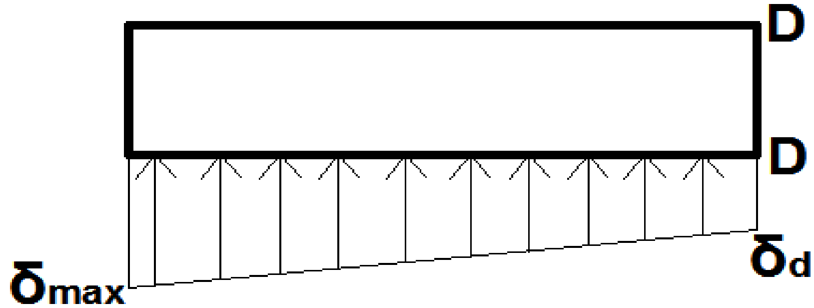
$$A_{Smin} = 1362mm^2/m$$

$$Spacing:\ S = \frac{A_b}{A_S} \times 1000 = \pi * \frac{\frac{20^2}{4}}{1362} * 1000 = 230.6mm$$

Use: **Ø20 c/c 230mm**

3. Design of the Heel:

a) At Section D-D



$$\delta_{max} = \frac{p}{b}\left(1 + \frac{6e_x}{b}\right) = \frac{878.64}{4.67}\left(1 + \frac{6 \times 0.03}{4.67}\right) = 196.51Kpa$$

$$\delta_d = \frac{3.11 \times (196.51 - 180.05)}{4.67} + 180.05 = 191.01Kpa$$

$$M_{D-D} = 1.56 \times 191.01 \times \frac{1.56}{2} + 0.5 \times (196.51 - 191.01) \times 1.56 \times \frac{2 \times 1.56}{3}$$
$$= \mathbf{236.88KNm}$$

$$V_{D-D} = 1.56 \times 191.01 + 0.5 \times (196.51 - 191.01) \times 1.56$$

$$= \mathbf{302.26KN}$$

$$D_{provided} = 1000mm$$

Check the effective depth for flexure:

$$D_{req.} \geq \sqrt{\frac{M_{D-D}}{R \times b}} \times 1000 = \sqrt{\frac{236.88}{1.692 \times 1000}} \times 1000 = 374.16mm$$

Use Ø24mm, Cover = 80mm

$$D_{req.} = d + 0.5\emptyset + cover = 374.16 + 0.5 \times 24 + 80 = 466.16mm$$

$$D_{avail.} = D_{provided} - 0.5\emptyset - cover = 1000 - 0.5 \times 24 - 80 = \mathbf{908mm}$$

$$D_{req.} = 466.16mm < D_{provided} = 1000mm \ldots\ldots\ldots\ldots Ok!$$

Reinforcement Calculation:

$$A_{Smin} = \rho_{min} \times b \times d = 0.0015 \times 1000 \times 908 = 1362mm^2/m$$

$$A_S = \frac{M_{c-c}}{f_s \times j \times d} = \frac{236.88}{218.4 \times 0.9 \times 0.908} = 1327.23mm^2/m$$

$$A_S\ used = 1362mm^2/m$$

$$Spacing:\ S = \frac{A_b}{A_S} \times 1000 = \frac{\frac{\pi \times 24^2}{4}}{1362} \times 1000 = 332mm$$

$\boldsymbol{Use\ \ \emptyset 24\ c/c\ 330mm}$

$Check\ the\ effective\ depth\ for\ Shear:$

$$Acting\ shear\ stress = \frac{V_{c-c}}{bd} = \frac{302.26}{1000 \times 908} = 0.33Kpa$$

$$Resisting\ shear\ stress = 0.0594\sqrt{24} = 0.29Kpa$$

$Acting\ Shear > Resisting\ shear\ stress$

$Minimum\ reinforecement\ shall\ be\ provided\ both\ in\ the\ vertical\ \&\ horizontal\ direction.$

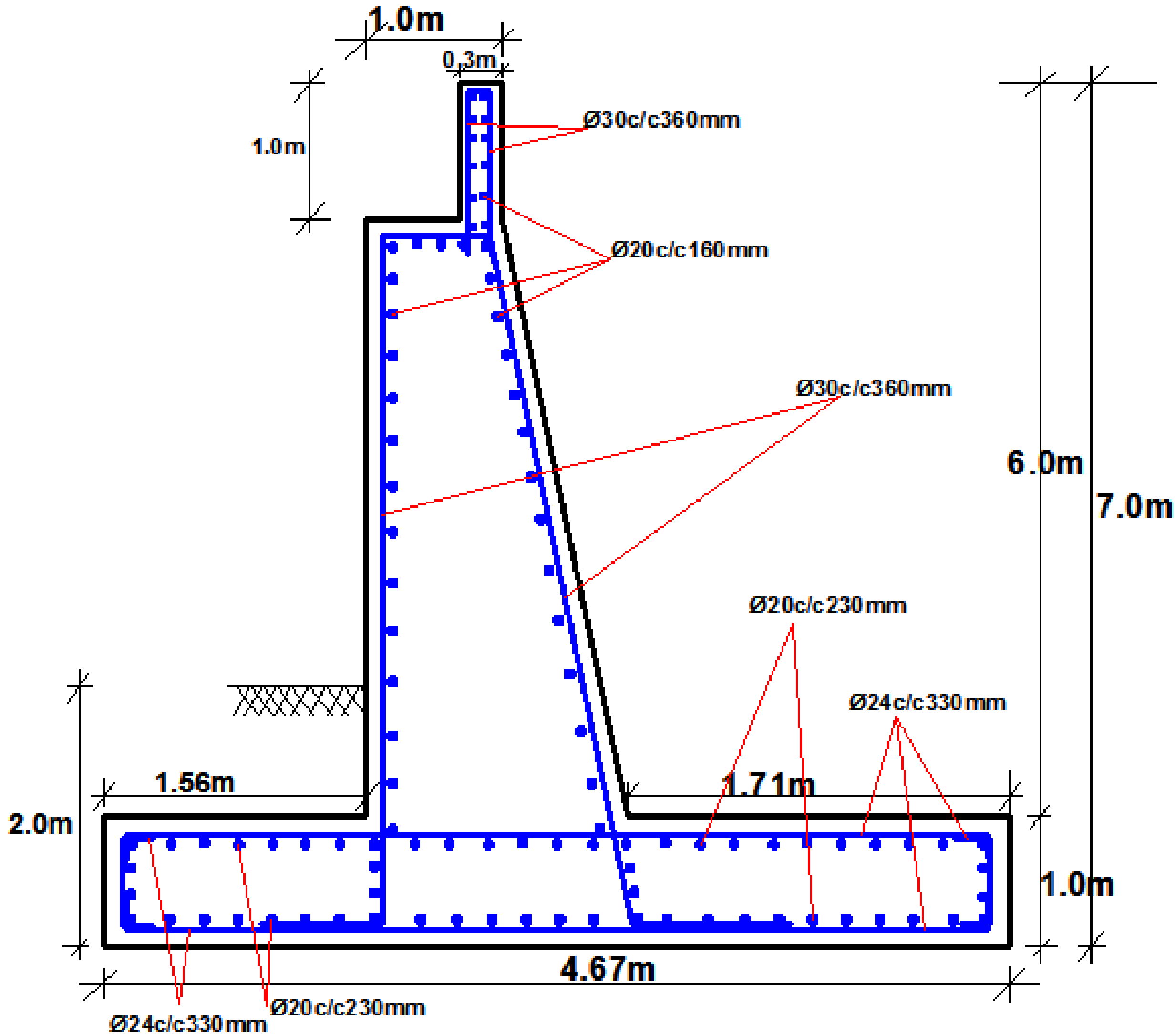
$Use\ \emptyset 20mm\ and\ concrete\ cover = 80mm.$

$$A_{Smin} = 1327.23mm^2/m$$

$$Spacing:\ S = \frac{A_b}{A_S} \times 1000 = \pi * \frac{\frac{20^2}{4}}{1327.23} * 1000 = 236.7mm$$

Use: Ø20 c/c 230mm

Reinforcement Detail Retaining Wall:



Appendix 2: Oral Interview Questionnaire for pedestrians

First of I would like to thank you for giving me time and patience to answer the questions.

My name is Biruk Desta, MSc in Civil Engineering under structural Engineering Student in the Addis Ababa University. I am carrying out a thesis research on "Provision of Underpass Pedestrian Crossings in Addis Ababa Railway Construction Projects". The data or information provided towards this research will be treated with utmost confidentiality and will be used for academic purposes only.

Date of Interview: ____

Q_1. Are you resident of Addis Ababa city?

A. Yes ☐ B. No ☐

Part 1: Pedestrian profile

Gender

1.0 Male ☐

2.0 Female ☐

Q_2. What is your age Group?

A. 17-30 years ☐

B. 31-50 years ☐

C. 51-66 years ☐

D. Greater than 66 years ☐

Q_3. Interviewee`s Physical Condition

A. Able Bodied ☐ B. Physically Disabled ☐

Part 2: Origin and destination

Q_4. Trip Purpose

A. Business ☐

B. Education ☐

C. Recreation ☐

D. Return Home ☐

Q_5. If you have used the road along AALRT, Have you ever crossed the AALRT road?

A. Yes ☐ B. No ☐

Q_6. If you have cross the road, Have you ever faced (observed) any problem when you cross the AALRT road?

A. Yes ☐ B. No ☐

Q_7. What Kind of problems you have faced when crossing the AALRT road?

A. Walking long distances along the AALRT to find Pedestrians crossing.

B. Crowdies of traffic flow on the road (Congestion).

C. Long waiting time to Cross the road (time consuming).

D. Jumping over the Safety Fence attempting to pass the AALRT road shortly. (Exposing life in Danger).

E. All

Please state any comment you would like to make on this survey and/or issues addressed

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THANK YOU VERY MUCH FOR GIVING ME YOUR TIME AND PATIENCE TO ANSWER THE QUESTIONS

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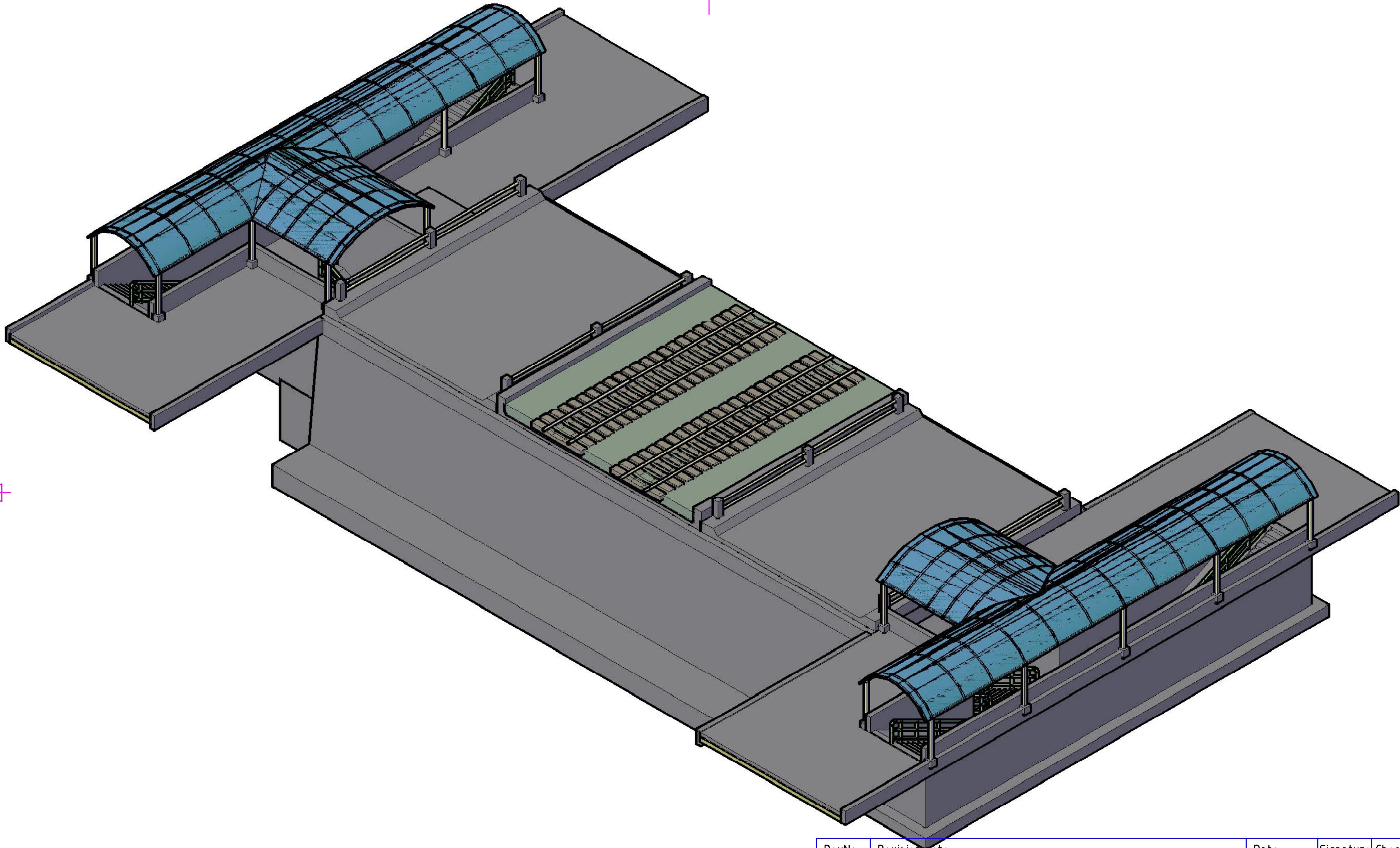

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Itemref	Quantity	Title/Name, designation, material, dimension etc			Article No./Reference		
Designed by Biruk D.		Checked by Dr. Asnake A.	Approved by - date	File name Isometric	Date 24/04/2016	Scale 1:100	

Underpass Pedestrian Crossing

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Itemref	Quantity	Title/Name, designation, material, dimension etc	Article No./Reference	

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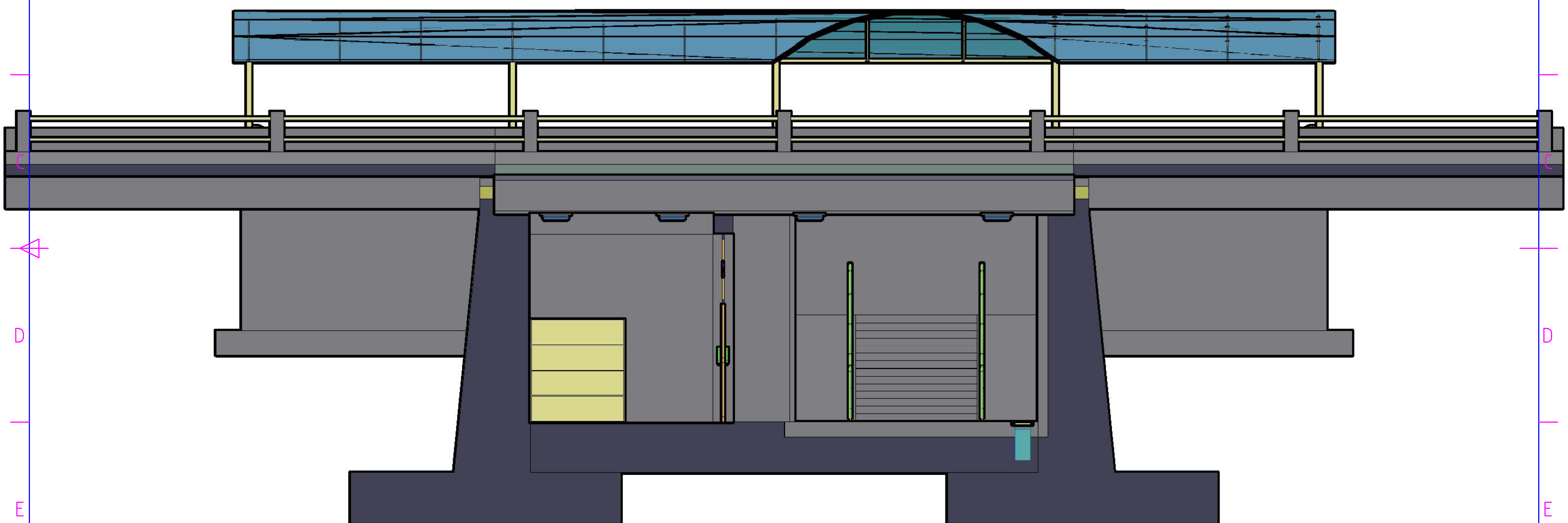
Underpass Pedestrian Crossing

DRW No. 03/04	Edition	Sheet 01/01



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Itemref	Quantity	Title/Name, designation, material, dimension etc		Article No./Reference		
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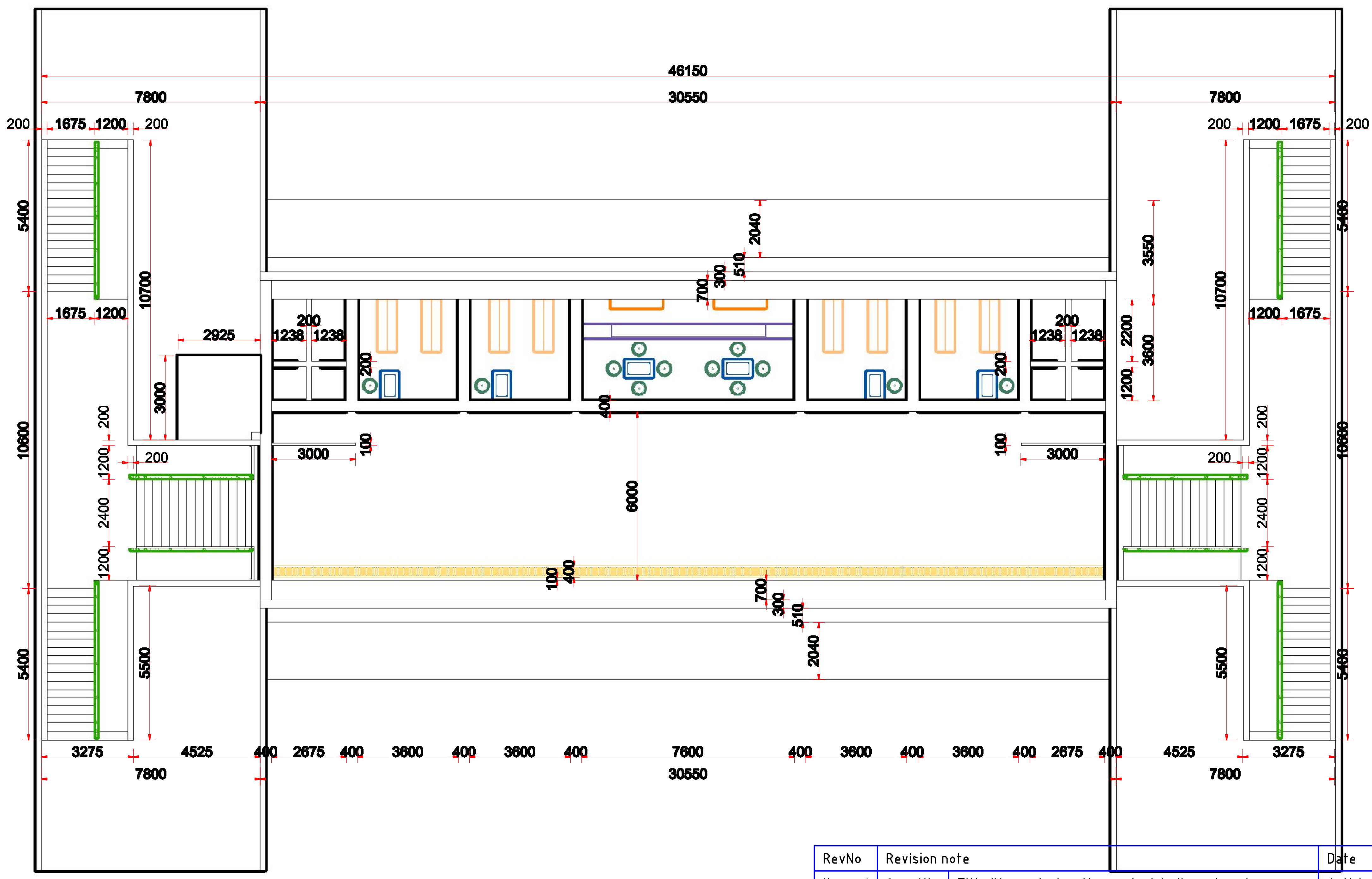
Underpass Pedestrian Crossing

DRW No. 04/04	Edition	Sheet 01/01



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Itemref	Quantity	Title/Name, designation, material, dimension etc		Article No./Reference		
Designed by Biruk D.	Checked by Dr. Asnake A.	Approved by - date	File name Floor Plan	Date 24/04/2016	Scale 1:100	

Underpass Pedestrian Crossing

DRW No. 02/04	Edition	Sheet 01/01