INSTITUTE OF TECHNOLOGY

A Teaching material on Railway Engineering

(CEng-5242)

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OUTLINE

<u>Text Books:</u>

1. **Bonnett, Clifford F.** *Practical Railway Engineering 2nd Edition.* S.L. : Imperial College Press, 2005.
2. **Profillidis, V.A.** *Railway Engineering Second Edition.* S.L. : Cambridge University Press, 1995.
3. **Chandra, Satish.** *Railway Engineering Second Edition.* S.L. : Oxford University Press, 2013.
4. **Selig, Ernest T.** *Track Geotechnology and Substructure Management.* S.L. : Thomas Telford Publications, 1995.

PREFACE

This teaching material is intended for use in the course railway engineering. It can be taken as an immediate guide for undergraduate students; and as an introduction to post graduate students whom do their projects work on railway related issues.

The material presented here can easily be completed in one semester, with a schedule in the above outline, and all modules should be covered.

The examples and exercises, in each module, were carefully organized to emphasize once again the information given and help to widen insights towards practical cases. In addition to this, objective based semester project is prepared.

Generally, in doing this work, international railway standards in conjunction with current Ethiopian Railway Corporation (ERC) practices are integrated to the course curriculum. Beside these, various theories and design books, thesis works, and journals are used to enrich as well as disclose clearly the fundamental codes' intention, explanatory figures and diagrams. The existing contents in the curriculum were rearranged in such a way that will let good coherence and sympathetic.

All Tracks Advance to Love and Peace!

MODULE-ONE

1. RAILWAY INFRASTRUCTURE

1.1. Definition

Rail transport: refers to the land transport of people or goods along guided paths called railways. A railway consists of two parallel rail tracks at a fixed distance (gauge) apart, usually made of steel and mounted upon cross beams called ties or sleepers.

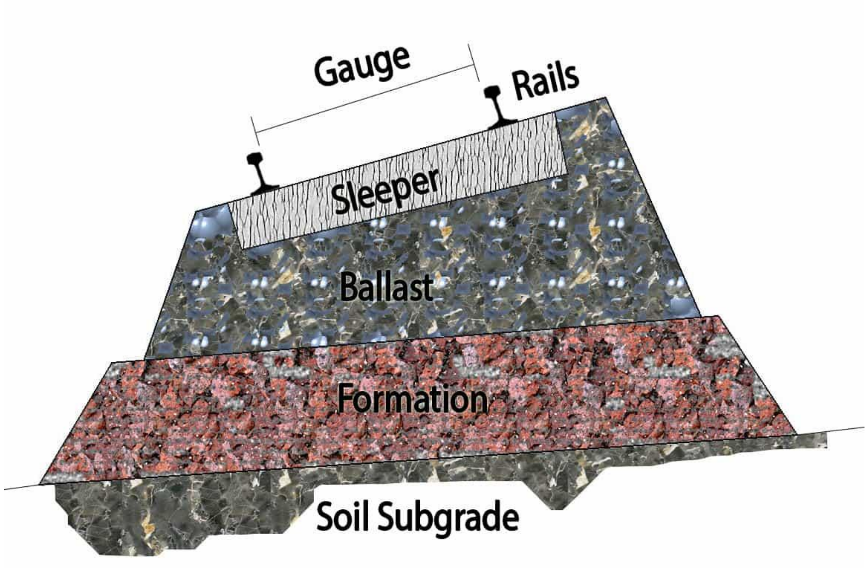


Figure 1-1: railway track components

Rail transport is also known as train transport. It is one of the most important, commonly used and very cost effective modes of commuting and goods carriage over long, as well as, short distances. Since this system runs on metal (usually steel) rails and wheels, it has an inherent benefit of lesser frictional resistance which helps attach more load in terms of wagons or carriages. This system is known as a train. Usually, trains are powered by an engine locomotive running on electricity or on diesel. Complex signaling systems are utilized if there are multiple route networks. Rail transport is also one of the fastest modes of land transport.

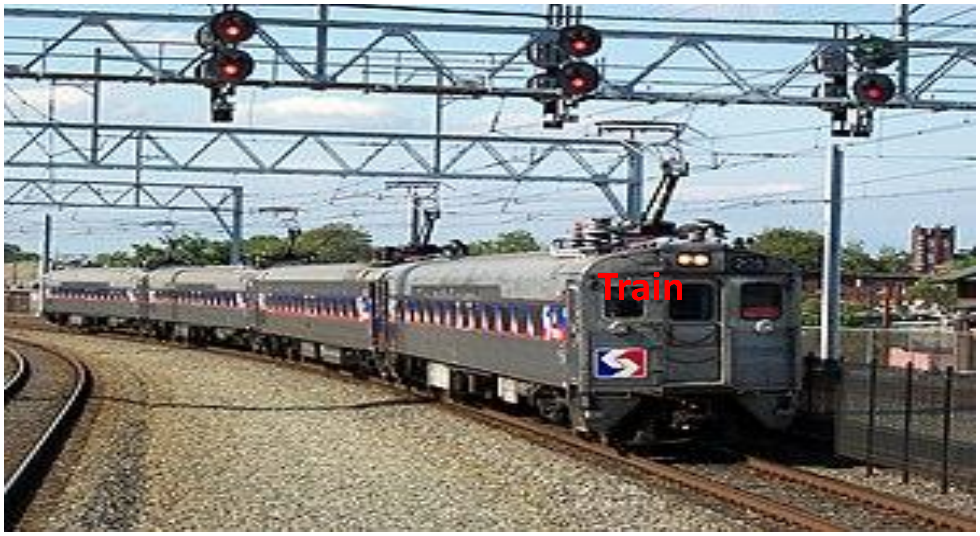


Figure 1-2: Rail transport (typical)

Rail transport has emerged as one of the most dependable modes of transport in terms of safety. It has fixed routes and schedules. Its services are more certain, uniform and regular compared to other modes of transport. Rail transport originated from human hauled contraptions in ancient Greece. Now it has evolved into a modern, complex and sophisticated system used both in urban and cross-country (and continent) networks over long distances.

1.2. Railway History of the World

In medieval times people mostly travelled by foot or horseback and any form of transportation was mainly for moving goods.

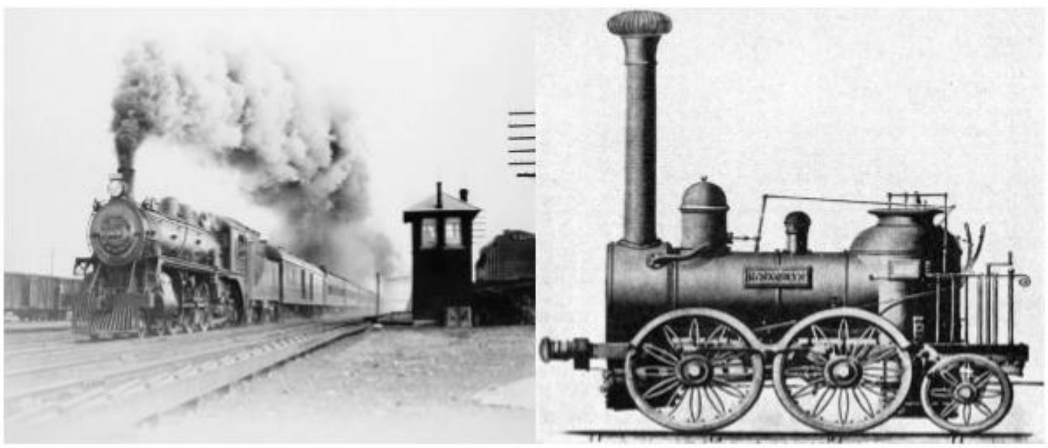
The first railways were laid down in the seventeenth and eighteenth century for horse drawn trains of wagons in collieries and quarries. These ‘hauling ways’ initially had a surface of stone slabs or timber baulks, which soon proved unsatisfactory as the loads carried inevitably grew heavier.

As the Industrial Revolution progressed, the idea was developed further by adding cast iron or wrought iron plates to reduce wear on the wooden baulks. This evolved further to iron edge rails enabling the use of flanged wheels for the first time.

By the time steam locomotives came on the scene, in the early nineteenth century, wrought iron rails and later steel rails were developed which were strong enough to support these heavy axle loads without assistance from longitudinal timbers.

The 'great' or 'golden' age of railways in the sense that they virtually monopolized inland transport was over in most countries by World War I. After a mid of 20th century rationalization of the rail network began and the following activities were taken to compete with other massive mode of transportation.

- Steam locomotives were avoided and replaced with full electrification of the lines
- Modernization of passenger coaches
- Safety system upgrading
- Freight wagons were also highly improved with the introduction of high capacity and with box containers and fully braked units.



(a) (b)

Figure 1-3: Railway transportation in the early 19th century (a) freight train (b) steam locomotive

In recent years, railway systems have played a significant role in transportation systems due to the demand increase in conveying both cargo and passengers. Due to the harsh environments and severe loading conditions, caused by the traffic growth, heavier axles and vehicles and increase in speed, railway tracks are at risk of degradation and failure. Condition monitoring has been widely used to support the health assessment of civil engineering structures and infrastructures. Railway reforms are still very much in progress in many countries. One of the major objectives driving these reforms has been to ensure that end-user prices are at an efficient level (considering the level of costs and the price of substitute services), productive efficiency is high (and therefore subsidies are low), and investment and innovation guarantee a satisfactory level of service quality, safety and variety.

1.3. Rail Transport in Africa

Almost all of the rail systems in Africa have their origins in the early 20th century when European colonial powers built railway lines to support military movements and to transport goods produced in the large mining or farming operations.

After the continent achieved independence, railway networks were broken up according to the new national borders, thus in some cases reducing their markets and economies of scale. This led to a situation of severe deterioration sometimes near a point of no return.

With a few exceptions (mainly in the RSA and Northern Africa), African railways clearly lag behind those of most other regions in the world. Rail transport has faced the same constraints and challenges as elsewhere. But, poor economic, technological and institutional conditions have further aggravated the situation in Africa. The result is outdated infrastructure, sometimes approaching a point of no return. The operations are clearly below international standards.

Concessions introduced in the 90s, under the impulse of the World Bank and other international donors, have halted the declining trend that threatened to dismantle many rail lines. But, the entire initiative has produced mixed results: in some cases it was a blatant failure and in quite a few others, if any, it was an outright success.

At present, Africa's rail network spans 65,000 km, of which the Sub-Saharan African (SSA) region accounts for 35.5 per cent, South Africa for 40.7 per cent and North Africa 23.8 per cent. Country-wise, South Africa has the largest rail network of around 22,051 km.

The growth of large cities, the opening of new mines and the strengthening of interregional corridors are some of the factors that will drive the commitment to rail during the 21st century.

1.4. Railway History of Ethiopia

Ethiopia is one of the landlocked Countries in African continent and shares its national borders with Sudan on the west, Kenya on the south, Somalia on the East, Djibouti on the North – east and Eritrea on the North; and also the place where origin and development history of humanity is traced back.

The railway infrastructure was first introduced to Ethiopia during the reign of Emperor Menelik II; who is the 'Father of Freedom' for black race. The line was jointly owned by the government of Ethiopia and Djibouti; and operated by CDE (*Chemin de fur Djibouti Ethiopien)*

Construction of the railway started at Djibouti in 1987. By 1902, the track reached Dire Dawa. However, it was not extended to Addis Ababa until 1917. This single track has been more than 100 years old one meter gauge railway; with a total length of 781 km (681 km in Ethiopia and the remaining 100 km in the territory of Djibouti) connecting Addis Ababa to the Port of Djibouti.

The maximum permitted speed for auto passenger trains was 85 km/hr. For Freight trains and for standard passenger trains, it was 50km/hr. The maximum axle loading on 20kg/m rail was 14metric tons. 18 metric tons was permitted on rails of 30kg/m. the maximum bridge loading was 17 tons.

Addis Ababa lost railroad access to the sea in 2004; and the construction of a 5,000 km railway network has been launched to link the capital to various regions of the country which was part of the country's five-year transformation plan in 2011.

Up-to-date, route-1&Addis LRT (phase-I) are under operation; and route-5 is under construction. But, constructions of the rest planned routes are not commenced yet.

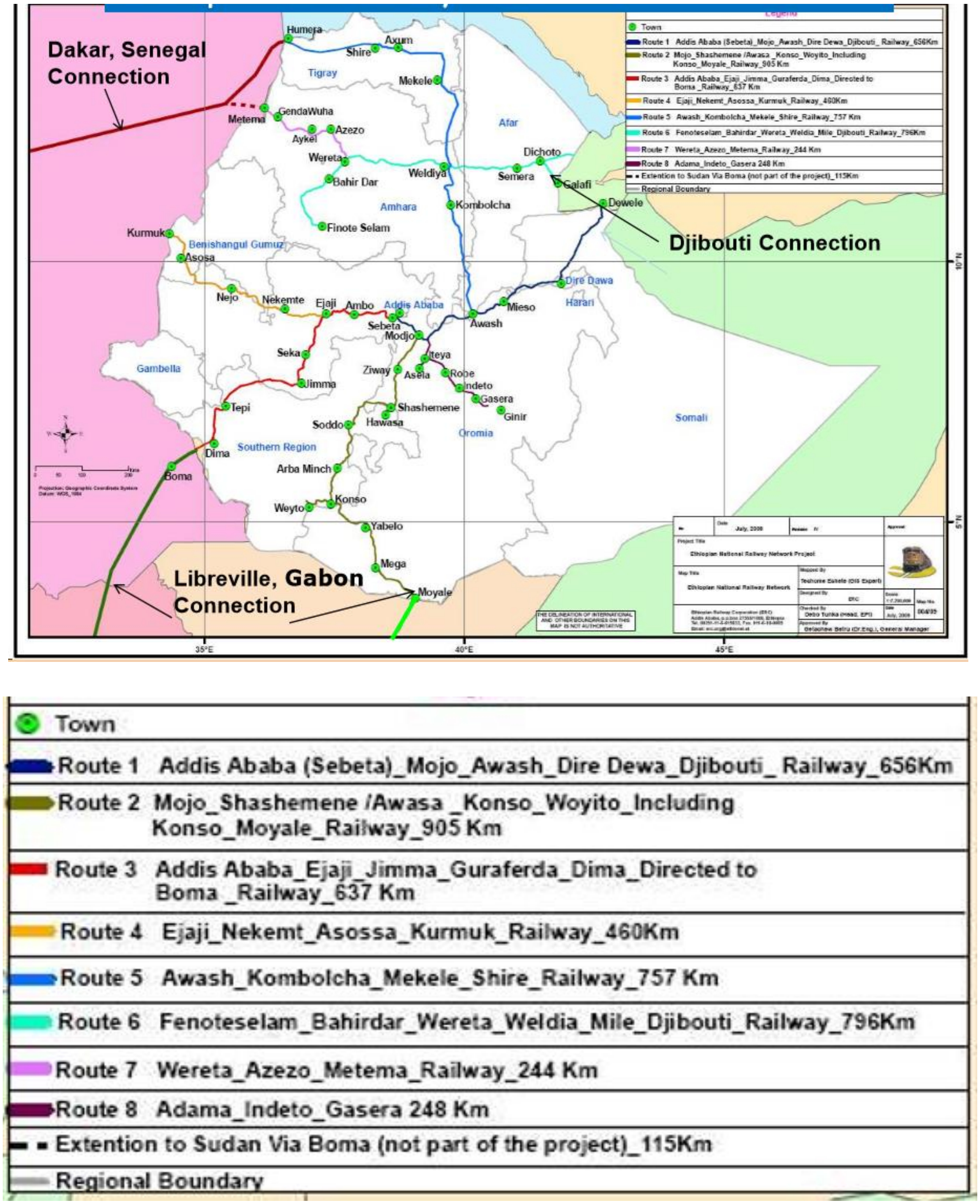


Figure 1-4: Ethiopian national railway network and African Connection

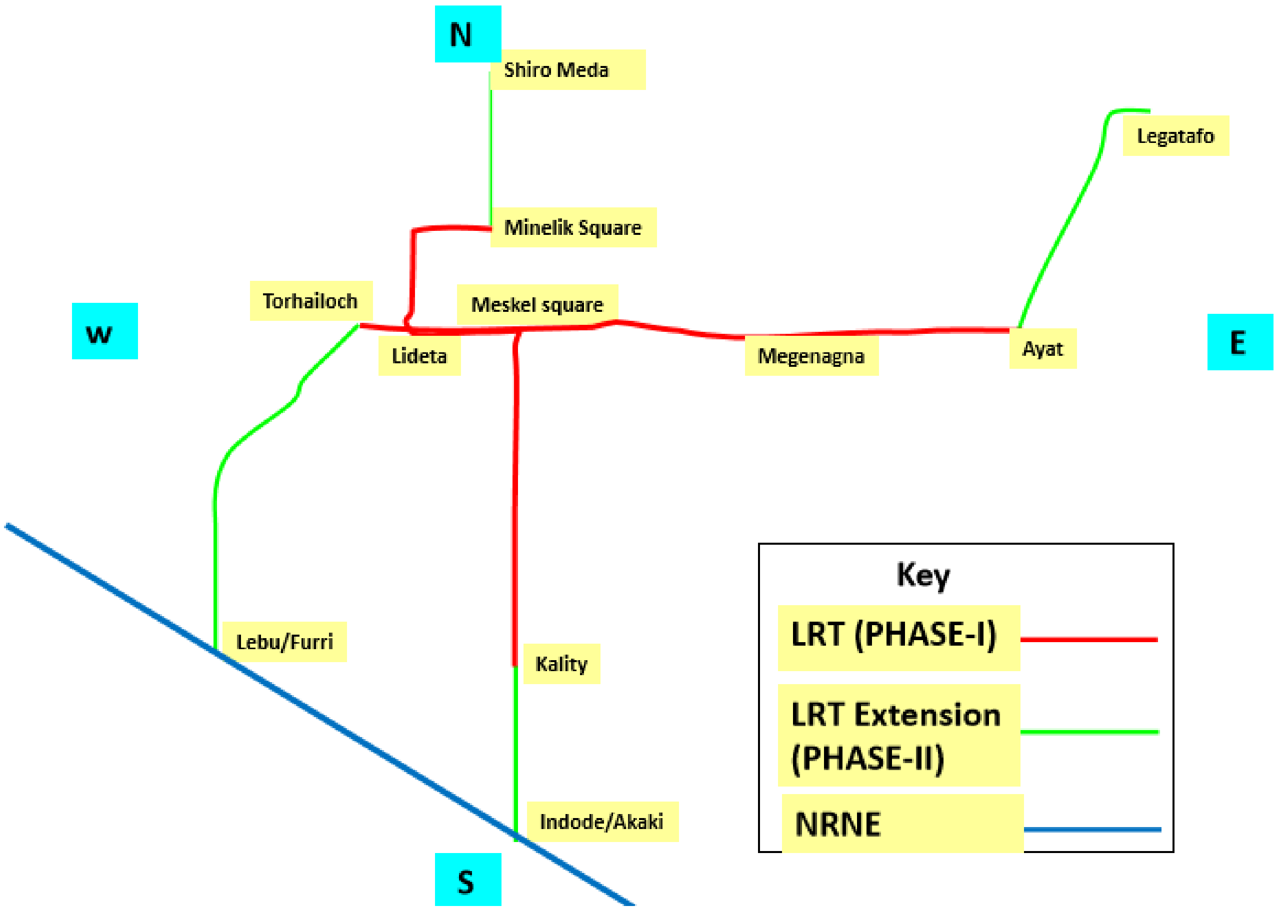


Figure 1-5: Addis Ababa LRT alignment with Extension

1.5. Components of Railway

The railway constituting conventional rail system may be divided into subsystems of either structural areas or operational areas. The structural parts consist of infrastructure, energy, control, command and signaling, management of traffic operation, and rolling stock. Whereas the operational concerns of railway subsystem consist of maintenance and telematics applications for passenger and freight services.

Figure 1-6: Components of Railway

In addition, infrastructure subsystem of the railway consist of tracks, points, engineering structures (bridges, tunnels, etc.), associated station infrastructure (platforms, zones of access, etc.), safety and protective equipment.

Figure 1-7: permanent infrastructure of railway

Track geometry, consisting of several parameters, is a significant factor influencing the ride quality and derailment risks. It describes the position that each rail, or the track centerline, occupies in space. By projecting the track geometry into various planes, track geometry can be specified. Track geometry parameters can be grouped according to the plane they reside in. The main parameters defining the track geometry are gauge (track plane), profile (longitudinal vertical plane), alignment (horizontal plane), cross level or super elevation (transverse vertical plane).

Rail track is a fundamental part of railway infrastructure and its components, for ballasted track, can be classified into two main categories: superstructure and substructure. The most obvious parts of the track as the rails, rail pads, concrete sleepers, and fastening systems are referred to as the superstructure while the substructure is associated with a geotechnical system consisting of ballast, sub-ballast and subgrade (formation). Both superstructure and substructure are mutually important in ensuring the safety and comfort of passengers and quality of the ride.

1.6. Overall Planning

In the railway field, planning the maintenance and renewal strategy from Life Cycle Cost (LCC) perspective gets more and more attentions recent years. The new approach looks at all the costs through the infrastructure life span and use the annuity (continuing payment with a fixed total annual spending) to evaluate the project alternatives. The comparison result can identify the most cost-efficient solution in a long run and therefore reduce the overall costs.

Additional, the most optimization today are dealing with the trade-off between renewal and maintenance. It is based on the analysis that the LCC yearly spends goes down first and then up by the increased maintenance. The life time of track can’t be infinitively extended through more maintenance. So there exists a LCC minimum yearly spends Point A as shown in the following figure.

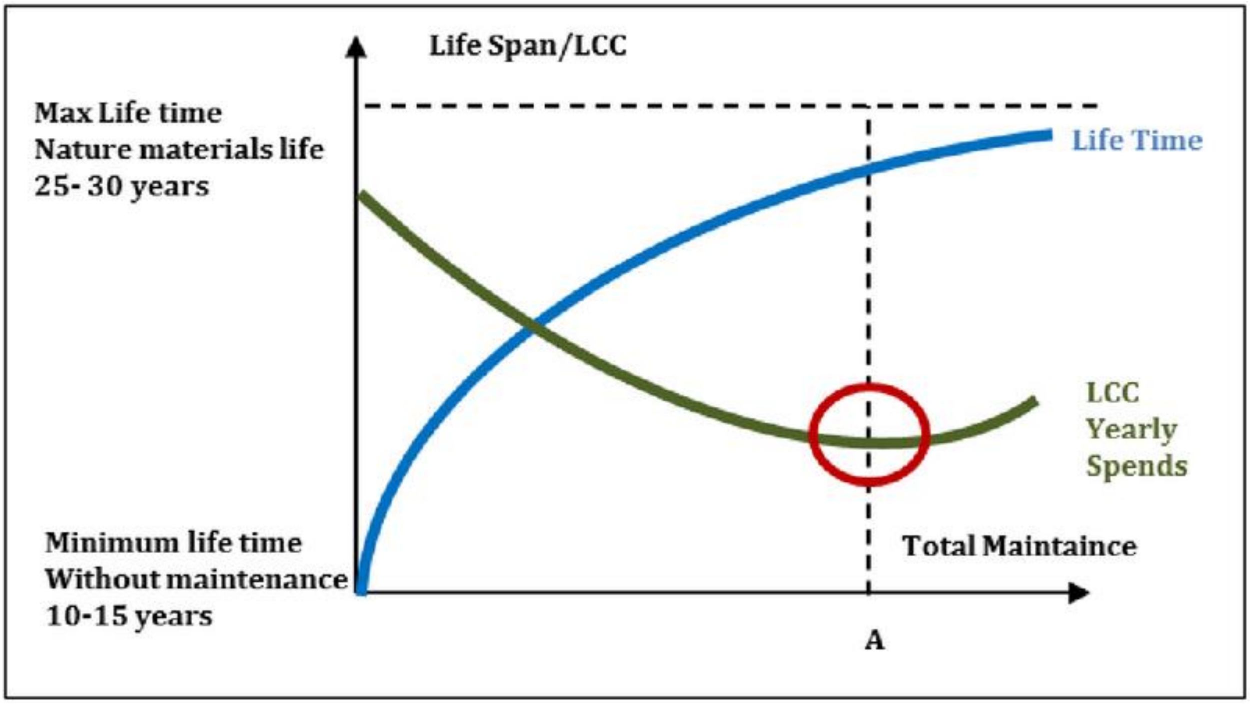


Figure 1-8: Optimizing the Maintenance Strategy (R. Li, A. Landex, O. Nielsen, 2013)

In constructing and maintaining any railway system infrastructure, there always are many activities which are carried out in each of the engineering disciplines which overlap onto other disciplines/activities. This is inevitable; that proper co-ordination and co-operation is necessary here. Beyond these, careful investigations on the features of rail transport system has a remarkable impact towards optimum planning.

There are several features that make railways different from other types of infrastructure.

a. **Rigidity of the railways:** From a technical point of view, railway is a mode of transport that presents substantially greater technical constraints than other modes.
   - Complex manoeuvres and/ or specific infrastructure are required to overtake and change direction and it requires shallow gradients and curves than roads.
   - Unlike roads or ships, services have to be controlled under a slot (path) system together with a communication system.
   - Trains are slower to react to unexpected events, e.g. it takes more time for a freight train to stop to avoid collision than a truck, and thus stricter safety regimes need to be in place.
   - Trucks, planes and ships have far more flexibility to change their route if required for commercial reasons or in the event of an incident, compared with trains.

b. **Need for an operator:** Although both railways and roads have infrastructure, which has to be maintained, anyone with a truck or car can use a road whereas for railways you need a specific operator who has to make a significant investment in rolling stock before they can operate. Private operators will not commit to an operation unless there is a sufficiently large market and if the operator does not have sufficient business the burden of cost will fall to the Government.
c. **Interoperability:** Planes and ships can move freely at ports and airports around the world. The main technical constraints are those related to dimension e.g. draught or wingspan. The same happens with road transport, a truck can theoretically move along any road with little technical limits. Gauge width is potentially the most significant constraint to interconnect railway networks, but other aspects such as electrification, communications systems, maximum axle load, train length or tunnel clearances are also factors on many networks.
d. **An alternative to other means of transportation:** Railways are only one of the options for moving people and goods in a competitive transportation framework i.e. road in the freight and passenger segments and with air in the long distance passenger segment. The user chooses the mode of choice according to cost, travel time, availability of service, comfort and reliability among other criteria.
e. **Impact of infrastructure constraints on the overall standards of service and performance:** The overall standards of service in a railway, however long it is, will be determined by the standards on the worst section in the line.

Example 1-1: *If along 1,000 km line there is a 10 km section where only 10 tonnes per axle are allowed, trains covering the whole distance will generally be limited to 10 t per axle, regardless that higher tonnage is possible along the rest of the track. In this case upgrading the 10 km section will bring a great positive impact along all the line. Also, the lowest clearance of all tunnels along a railway line will limit the overall vertical clearance.*

Depending on relative costs in each area, an optimum programme could be drawn up jointly which would keep overall costs and disruption to a minimum in the long term.

Table 1-1: Comparing railways external costs with other means of transportation

Externalities	**Main benefits from using railways versus other means of transportation**
1. Accidents	Railways may substantially reduce road casualties and their related costs. - *The cost of accidents is 50 times less for rail than road traffic.* - *It is estimated that by 2030 road traffic accidents in developing countries could be as high as AIDS deaths.* - *Developing mass transit systems in urban areas could reduce such costs by mode transfer from road to rail.*
2. Air pollution/ climate change	Railways produce far less air pollution than other modes. - *Air pollution causes health costs, crop losses and building damage.* - *Rail diesel produces 50% more air pollution than electric rail although this is still significantly less than car.* - *In non-urban areas, railways produce 3 times less air pollution costs than road freight.*
3. Noise	Railways produce less noise costs than road modes. - *Noise causes health costs and general annoyance*
4. Up-and downstream processes	The external cost is higher for rail than for other modes. - *Such costs represent climate change effects and air pollution, derived from fuel and electricity production.* - *Waterborne freight transport can have the least impact.*
5. Others	Railways perform best than any other mode, except aviation. - *Nature and landscape costs, biodiversity losses, soil and water pollution recovery costs, and times losses for non-motorized users in urban areas.*

1.7. Railway Classification

Railway can be classified on the basis of the following parameters: axle load of rolling stock, maximum running speed, volume of passenger and goods traffic, designed speed, and Significance of railway construction.

For instance, in china, railways are classified in to three categories based on the annual volume of passenger and freight traffic, role they played in railways network, and maximum design speed. These are:

1. Railway line for passenger traffic

These are mainly responsible for transportation of passenger with design speed of passenger car not less than 200km/h. Further classified as:

a. High-speed railway :
   - Railways for passenger with maximum design speed of 250km/h and over and play trunk parts in railway network
   - Generally built in a developing region where is densely populated and has a heavy volume of passenger traffic.
   - It links political center and economic center, or economic center and economic center.

b. Rapid -speed railway
   - Railways which play a linking and auxiliary part in railway network for passenger traffic, with designed speed of 250km/h or lower.

2. Railway line for mixed passenger and freight traffic

These railway lines are responsible for passenger and freight transportation, with design speed of 160km/h or lower (passenger train) and 120km/h (freight train). Railways for mixed passenger and freight traffic can be classified as 4 grades,

a) *Class I Railway:* Play a trunk part in railway network, and the volume of short-term traffic is no less than 20 million tons.
b) *Class II Railway:* Play a linking and auxiliary part in railway network, and the volume of short-term traffic is less than 20 million tons.

c) *Class III Railway:* Give service to a local zone, and the volume of short-term traffic is less than 20 million tons and larger than 5 million tons.

d) *Class IV Railway:* Give service to a local zone and the volume of short-term traffic is less than 5 million tons.

3. **Line for goods traffic:** heavy haul railway line

1.8. Railway Technical Standards

Specific rules and standards are necessary for railways to carry out safe, high-speed, punctual, and efficient train operation. Therefore, it is essential for the national government to clearly indicate standards on safety conditions and so forth so that railways can satisfy a specific level of social requirements by meeting the standards.

The main purposes of establishing the railway technical standards are:

- Ensuring of safety
- Maintaining of railway network and ensuring of railway transport characteristics
- Ensuring of convenience for users
- Environmental countermeasures
- Reduction of production cost

Railway technical standards are broadly classified into:

1. **Compulsory Standards:** Which stipulate safety and so forth;

The governments and national railways of individual countries should established compulsory technical standards on railway construction and operation, for such purposes as ensuring of railway safety and maintaining of railway networks.

2. **Design Standards:** Which Complement the Compulsory Standards

To formulate the contents of the compulsory technical standards which stipulate such matters as safety, it is necessary to establish rules, from the stage of designing, concerning the strength of materials of facilities and rolling stock, safety level of structures, and so forth.

3. **Voluntary Standards:** Which aim at the enhancement of production efficiency, elimination of trade barriers, and so forth.

For industrial products, there are various international, regional, national, and group standards. There are similar standards for railway sectors as well, and each country is making efforts to adjust its regional, national and group standards to the national standards.

The voluntary standards for railway sectors are as follows.

a. ***International Standards***
   - ISO (International Organization for Standardization)
   - IEC (International Electrotechnical Commission)

b. ***Regional Standards***

The European Committee for Standardization (CEN) and the European Committee for Electrotechnical Standardization (CENELEC) have jointly established unified standards "EN (EURO NORM)" for European countries.

c. ***National Standards***

Individual countries have respective organizations for standardization, in order to promote standardization of products and so forth in various industrial sectors. These organizations have established national standards applicable to their countries.

d. ***Group Standards***
   - UIC (International Union of Railways)

Legislation of Railway Technical Standards

Compulsory standards must be adhered to in each railway company. For this reason, it is necessary for these technical standards to be clearly prescribed in the legal system. Since the technical standards established by the Government are the minimum performance standards necessary for ensuring safety and so forth, railway operators should decide, based on these standards, their own standards on the structure and maintenance of their specific railway facilities and rolling stock as well as on the handling of train operation.

Items of main technical standards

Main technical standards include the basic standards and types of railway facilities, which shall give obvious influence on the traffic capacity, construction cost, operation quality, and the selection of other equipment standards. The design of railway of mixed service shall include main technical standards of: numbers of main lines, ruling grade, minimum radius of curvature, available length of arrival & departure, kind of energy supply, types of locomotives, tonnage rating, locomotive routing, and type of blocking.

Main technical standards for railway of passenger traffic include: designed speed, distance between centers of main line tracks, minimum plane curve radius, and maximum gradient, available length of arrival and departure line, EMU type, train operation control mode, train operation command mode, and minimum head

EXERCISE 1-1

1. Compare railway, road, airway, and water transport systems; in terms of the following parameters.
    a) Reaction to unexpected events
    b) Maintenance requirement
    c) Operation requirement
    d) Ease of privatization
    e) Bio diversity loses
    f) Climate change
    g) Overall planning
2. What are opportunities and challenges for railway development in Ethiopia?
3. List necessary disciplinary specializations for optimum design, construction, and operation of railway components?
4. Discuss merit and demerit of being a member of international and group railway standards for; our beloved country, Ethiopia?
5. Discuss on differences in main technical standards, for urban and regional rail applications; (case of Ethio-Djibouti railway line and Addis LRT)?

MODULE-TWO

2. PRELIMININARY DESIGN OF RAILWAY LINE

Introduction

Preliminary design of railway is a part of the development phases of railway projects where all of the geometric design elements, including a preliminary estimate of the preferred design solution are documented for input to the detailed design stage.

Preliminary design report (PDR)

- Mainly consists of one or more solutions without necessary land and soil surveys where information is obtained from existing maps and plans,
- Ultimate goal of the PDR is to convince that it is worth pursuing to the next phase (detail design)
- Mainly includes **environmental impact assessment** and **feasibility study reports.**

2.1. Environmental Impact Assessment (EIA)

Transport is considered as unsustainable activity because of the infrastructure for transport operations, such as roads and railways, use non-renewable resources. Transport process is responsible for great pollution, damages irreplaceable resources and also leads to long-term environmental change.

There are several proclamations provided by the FDRE related to Environmental protection issues. For instance, the Constitution under Article 44 highlights about environmental rights as follows:

a) All persons have the right to a clean environment.
b) All persons who have been displaced or whose livelihoods have been adversely affected as a result of state programs have the right to commensurate monetary or alternative means of compensation, including relocation with adequate state assistance.

Therefore, a complete life cycle analysis (LCA) of railways, including the energy consumption during the operating phase should be practiced as a parameter for environmental optimization which is expected to achieve:

- The reduction of environmental global effects of infrastructures considering geometry and construction;
- The sparing of natural resources (including energy and water);
- The reduction of emissions (greenhouse gases, carbon monoxide (CO), lead (Pb), particulate matter (PM)...);
- The reuse of deconstruction materials in comparison to other possibilities;

The estimation of the damage to the environment is not an easy task because apart from the direct consequences (i.e. cost per CO2 tonne) there are also many side effects such as oil waste, noise pollution etc. The content of environmental impact assessment is dependent on compulsory standards of nations and other standards for which countries are a member. The approximate direct cost could be estimated via the carbon tax system.

2.1.1. Mitigation Measures

A construction of railway line would lead to a reduction of CO2 emissions, which is something positive, however its construction and operation will cause environmental disturbances also. This needs development of a method, how to mitigate the impacts, prior to designing the railway line.

Even though type and extent of mitigation measures varies with specific project characteristics, it’s most common to manage the impact within three phases of life cycle of railway infrastructure

I. **Construction phase:** including three stages of the life cycle; Production of materials (raw materials extraction, manufacturing and processing), materials and machines transport, and implementation required for the infrastructure.

II. **Operation and maintenance phases**: corresponding to the use of the infrastructure with train traffic and maintenance during life use phase;

III. **End of life:** At the end of life phase, the infrastructure is demolished. Demolition materials are transported, treated and recycled or stored.

In each phase, there should be alternative scenarios to be compared by taking sustainable construction method in mind and appropriate mitigation measures must be taken for;

- Loss of Land under Various Uses
- Air Pollution and Noise
- Competition for water resources
- Impacts on Flora and Fauna Resources
- Impacts on Sensitive Ecosystem Wetlands
- Environmental Health and occupational Diseases
- Impact on Traffic Safety and Control
- Exposure to HIV/AIDS and Other Sexually Transmitted Diseases (STDs), etc.

2.2. Feasibility Study for Railway Projects

Why undertake a feasibility study?

- To determine whether, and under what conditions, a project will be technically, financially and economically viable
- To demonstrate to potential donors that project will produce acceptable commercial rate of return (i.e.it is both technically and financially viable)
- To demonstrate to governments and to other stakeholders that project will achieve acceptable social, or economic, rate of return

The following tasks are most commonly expected to be undertaken in a feasibility study of railway projects as minimum requirement.

A. Demand analysis and traffic forecasts
B. Engineering analysis and technical specs
C. Capital cost estimation
D. O&M cost estimation
E. Financial analysis
F. Economic analysis
G. Sensitivity analysis

After completion of the above tasks, It is possible to have good information of deciding whether the construction of a proposed railway line is technically feasible, socially acceptable, economically viable, or not. Therefore, this section presents a guideline to conduct feasibility study of railway projects.

2.2.1. Demand Analysis and Traffic Forecasts

Any railway project feasibility study should contain a detailed chapter on demand analysis and forecasting.

- It should provide forecasts adapted to the characteristics of the project.
- The impact on the existing rail traffic, on the competing modes (diverted traffic) and the amount of traffic generated or induced by the project must be clearly identified.
- It will be necessary to distinguish between traffic categories that need to be treated differently.
- It is vital for accurate forecasting of cash flows in the Financial Analysis and of net economic benefits in the Economic Analysis
- Many projects have failed to realize the benefits expected of them simply because demand has fallen short of forecast levels
- Since railway assets are long-lived (50 years or more), demand forecasts in terms of freight tonnage/tonne-km and/or passenger volume should cover at least 20 operating years beyond the construction period
- Extensive sensitivity testing of financial and economic results should be carried out for variation of demand from forecast level

Demographic and economic factors are the basis for analyzing and projecting the demand for railway projects. However, analyzing these data to obtain meaningful information requires great concentration and devotion.

2.2.1.1. Demand Forecasting Methods

The purpose of demand forecasting is to use the best available information to guide future activities toward organizational goals. Forecasting methods can be classified as qualitative or quantitative.

a) **Qualitative methods**: generally involve the use of expert judgment to develop forecasts. Such methods are appropriate when historical data on the variable being forecast are either not applicable or unavailable.

b) **Quantitative forecast methods**: can be used when past information about the variable being forecast is available;

Majority of quantitative approaches fall in the category of time series analysis. Analysis of the time series identifies patterns. Once the patterns are identified, they can be used to develop a forecast. One of the best known quantitative demand forecasting approaches is simple linear regression method.

Example 2-1: *Conduct freight traffic forecasting of Ethio-Djibouti line for the next 50 years" time period with the following data (use simple linear regression method)*

Table 2-1: import and export of Ethiopian trade

Year	Import (tons)	Export (tons)
1997	1,106,794	373,818
1998	3,556,019	241,484
1999	2,553,123	247,883
2000	2,727,216	323,432
2001	3,369,679	350,832
2002	2,595,131	561,291
2003	5,092,941	506,009
2004	3,832,543	539,496
2005	4,413,599	748,154
2006	4,184,892	690,610
2007	4,955,151	831,931
2008	7,603,417	834,480
2009	8,218,723	909,833
2010	7,192,247	1,163,535
2011	7,391,318	1,215,005
2012	8,593,864	1,318,950
2013	7,533,856	1,373,225
2014	10,246,626	1,594,061
2015	10,444,944	1,551,748
2016	14,595,826	2,205,872

Table 2-2: port distribution based on ministry of transport

Harbor	2010 GC.	Five Years Later 2015 GC.
Djibouti port	93%	75%
Berbera port	5%	15%
Sudan port	2%	10%

Solution:

Step-1: coefficient of correlation "r"

In order to use regression as a forecasting model factors must be considered for accuracy of regression model like the coefficient of correlation "r" which explains the relative importance of the relationship between x (year) and y (import/export) and the one with r^2>95% will be selected.

$$r = \frac{n*\sum xy - \sum x \sum y}{\sqrt{\left[n\sum x^2 - \left(\sum x\right)^2\right] * \left[n\sum y^2 - \left(\sum y\right)^2\right]}}$$

Coefficient of correlation values

Import	r = 0.975048921	r^2 = 0.950720399	r^2 = 95.10 %
Export	r = 0.98230249	r^2 = 0.968800032	r^2 = 96.88 %

The value of coefficient of correlation "r" shows that there is a strong relationship between the variables x and y as year increase import and export increases simultaneously. The result r^2 indicates that it is possible to use linear regression to forecast the future freight demand.

Step-3: future demand using regression formula

$$y = ax + b$$

Where: - a, is the y intercept, b is the slope of the line, y is the tons and x is the year the coefficients a and b are calculated based on the following formula.

$$a = \frac{\sum y * \sum x^2 - \sum x \sum x * y}{n \sum x^2 - (\sum x)^2} \text{ and } b = \frac{n * \sum x * y - \sum x \sum y}{n \sum x^2 - \sum x \sum y}$$

Regression coefficients and equations of import and export

		Import	Export
coefficients	a	-1,048,367,144	-171,575,595
	b	525,481	85,948
Regression equation		y = -1,048,367,144 + 525,481* x	y = -171,575,595 + 85,948* x

Step-4: Djibouti port share from the total volume

The study time period is 50year; as a result the percentage of port share fluctuation is expected throughout these years. By assuming port share fluctuation will be insignificant and port of Djibouti share remains 75% throughout the study period.

$$djibouti\ port\ share = 75\% * total\ forcasted\ freight\ volume$$

2.2.1.2. Modal Split between Different Transport Modes

The objective of modal spilt is to carry out the ratio for different modes of transport. The selection of one mode or another is a complex process that depends on factors such as: travel time, cost, and Interval between departures.

The procedures for modal split of freight and passenger transport are different in many aspects like data availability, the complexity and nature of mode, the content and background of the models used and the details of practical application.

Proper modal split analysis involves the assessment of the capacity, cost and performance of existing and proposed alternative transport systems along the corridor. In spite of increased role of non-price factors in the modern transport marketing process, price still remains a critical element.

Binary logit model

- It is used to split the modes where the travel choice between different modes is made.
- The traveler will associate some value for the utility of each mode if the utility of one mode is higher than the other, then that mode is chosen.
- But in transportation, we have disutility also. The disutility here is the travel cost.

According to (UIC, 2012); this can be represented as:

2-1

$$c_{ij} = a_1 t^{v}{}_{ij} + a_2 t^{w}{}_{ij} + a_3 t^{t}{}_{ij} + a_4 f_{ij} + a_5 \varnothing_j$$

Where:

- $t^{v}{}_{ij}$- *is the in-vehicle travel time between i and j,*
- $t^{w}{}_{ij}$- *is the walking time to and from stops,*
- $t^{t}{}_{ij}$- *is the waiting time at stops,*
- f_{ij} *-is the fare charged to travel between i and j, and*
- $\varnothing_j$ *-is the parking cost.*
- *The constants are;* $a_1=0.03$, $a_2=0.04$, $a_3=0.06$, $a_4=0.1$, $a_5=0.1$

If the travel cost is low, then that mode has more probability of being chosen. According to (UIC, 2012);

2-2

$$P_{ij}{}^{1} = \frac{e^{-c_{ij}{}^{1}}}{\sum e^{-c_{ij}{}^{m}}}$$

Where: P_{ij} *is the percentage mode choice;* c_{ij} *the generalized cost.*

NOTE: Multinomial logit model can be used for more than two alternatives

Example 2-2: *Modal split of freight from city* ***A*** *to city* ***B.***

Input data;

- *distance from* ***A*** *to* ***B****: for train- 113km, for truck- 98km*
- *Average speed:for train 80km/hr, for truck 60km/hr.*
- *in-vehicle travel time: for train- (113km)/(80km/h) = 1.4125hr= 84.75min; for truck- (98km)/(60km/h) = 1.633hr= 98min*
- *the walking time to and from stops: for train-5 min, for truck-5min*
- *the waiting time at stops: for train-2 hours, for truck-3 hours*
- *the fare charged to travel: for train-0.046USD/km, for truck-0.047USD/km*

Step-1: generalized cost parameter c_{ij},

$$c_{ij(train)} = 0.03*84.75+0.04*15+0.06*2+0.1*0.046+0.1*0=3.672$$

$$c_{ij(truck)} = 0.03*98+0.04*5+0.06*3+0.1*0.047+0.1*4=3.725$$

Step-2: The probability of choosing on mode,

$$P_{ij(train)} = 42\%$$

$$P_{ij(truck)}=58\%$$

Therfore, fourty two percent of the total freight demand is expected to have a train as first choice of travel from ***A*** *to* ***B****.*

2.2.2. Engineering Analysis and Technical Specs

Basic technical specifications (e.g. maximum or ruling gradient, minimum curvature, axle loading, loop lengths, station spacing's, speeds and signaling requirements) should be established in advance of the capital cost estimation.

The need for proper geometric design of a track arises because of the following considerations:

- To ensure the smooth and safe running of trains
- To achieve maximum speeds
- To carry heavy axle loads
- To avoid accidents and derailments due to a defective permanent way

- To ensure that the track requires least maintenance
- For good aesthetics

Basic geometric elements of railway consist of alignment, transition length, track gauge, and track cant or superelevation.

2.2.2.1. Track Gauge

The gauge is the distance between the inner sides of the head of rails measured 5/8 inches below the top of rails; and can be chosen with the following theoretical considerations

a) **Cost considerations:** There is a proportional increase in the cost of acquisition of land, earthwork, rails, sleepers, ballast, and other track items when constructing a wider gauge.
b) **Traffic considerations:** As a wider gauge can carry larger wagons and it can theoretically carry more traffic.
c) **Physical features of the country:** It is possible to adopt steeper gradients and sharper curves for a narrow gauge as compared to a wider gauge.
d) **Uniformity of gauge:** The existence of a uniform gauge in a country enables smooth, speedy, and efficient operation of trains.

Table 2-3: various rail gauges adopted by different railways in the world (Handan, 2009)

Type of Gauge	Gauge (mm)	% of total length	Countries
Standard gauge	1435	62	England, USA, Canada, Turkey, Persia, and China
Broad gauge	1676	6	India, Pakistan, Ceylon, Brazil, Argentina
Broad gauge	1524	9	Russia, Finland
Cape gauge	1067	8	Africa, Japan, Java, Australia, and New Zealand
Meter Gauge	1000	9	India, France, Switzerland, and Argentina
23 various other gauge	Different Gauge	6	Various countries

In Ethiopia, the standard track gauge is used. The standard track gauge is 1435 mm and for this gauge the distance between the points of contact of the mean wheel circles with the rails, tack width S is 1500 mm

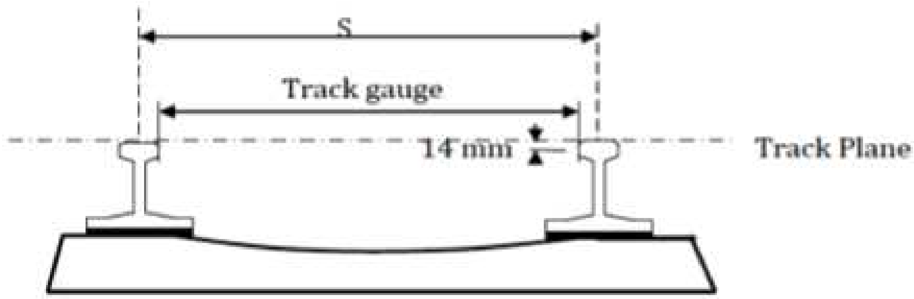


Figure 2-1: standard track gauge (Lindahl, 2001)

2.2.2.2. Alignment

The route upon which a train travels and the track is constructed is defined as an alignment. An alignment is defined in two fashions.

- First, the horizontal alignment defines physically where the route or track goes (mathematically the XY plane).
- The second component is a vertical alignment, which defines the elevation, rise and fall (the Z component).

Alignment considerations weigh more heavily on railway design versus highway design for several reasons.

- First, unlike most other transportation modes, the operator of a train has no control over horizontal movements (i.e. steering). The guidance mechanism for railway vehicles is defined almost exclusively by track location and thus the track alignment. .
- Secondly, the relative power available for locomotion relative to the mass to be moved is significantly less than for other forms of transportation, such as air or highway vehicles.
- Finally, the physical dimension of the vehicular unit (the train) is extremely long and thin.

Where a route between two points must be constructed; one option is to construct a shorter route with steep grades. The second option is to build a longer route with greater curvature along gentle sloping topography. The challenge is for the designer to choose the better route based upon overall construction, operational and maintenance criteria.

a) The designer should also add to the decision model environmental concerns, politics, land use issues, economics, long-term traffic levels and other economic criteria far beyond what has traditionally been considered.
b) The designer will have to work with these issues occasionally, dependent upon the size and scope of the project.
c) On a more discrete level, the designer should take the basic components of alignments, tangents, grades, horizontal and vertical curves, spirals and superelevation and construct an alignment, which is cost effective to construct, easy to maintain, efficient and safe to operate.

Circular curve

The most distinguished parameter for a circular curve is the radius, **R** constant and is related to the center of track, which is inversely proportional to curvature, **k** (Esveld, 2001)**.**

$$k = \frac{1}{R} \tag{2-3}$$

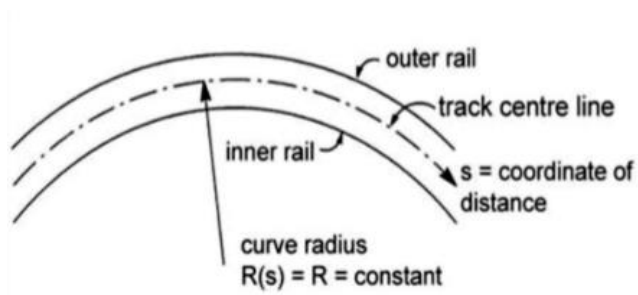


Figure 2-2: radius at center of track (Lindahl, 2001)

Curve radius is often expressed in terms of the angle α, in degrees, which is subscribed by a 100ft or 30.48m curve length (Esveld, 2001)

2-4

$$\alpha = \frac{1746}{R}$$

As a vehicle traverses a curve, the vehicle transmits a centrifugal force to the rail at the point of wheel contact. This force is a function of the severity of the curve, speed of the vehicle and the mass (weight) of the vehicle. This force acts at the center of gravity of the rail vehicle. This force is resisted by the track. If the vehicle is traveling fast enough, it may derail due to rail rollover, the car rolling over or simply derailing from the combined transverse force exceeding the limit allowed by rail-flange contact.

A vehicle running at a speed, **v** in a curve with a radius **R** undergoes a centrifugal lateral acceleration (Chandra, 2013) a_1

2-5

$$a_y = \frac{v^2}{R}$$

2.2.2.3. Track Cant

Cant is the difference between the levels of the two rails in a curve and is arranged to compensate part of the lateral acceleration by the gravity component.

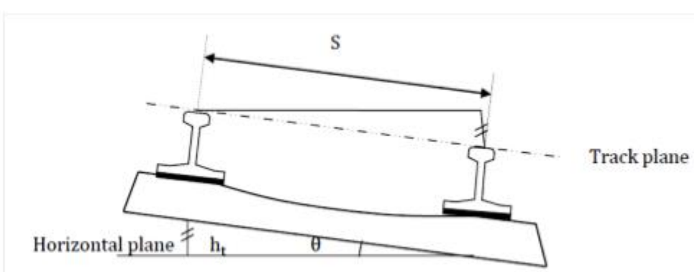


Figure 2-3: track cant (Lindahl, 2001)

Where: S = canter to center length of track gauge, h_t= cant height and θ=cant angle

The curve with a small radius must have cant in order to reduce or eliminate this lateral acceleration when it is not possible to make a suitable large curve radius.

A maximum value is set for cant because of the following problems which arise if a train is forced to stop or run slowly in a curve:

- Passenger discomfort
- Possible displacement of wagon loads
- Risk of derailment of freight trains in sharp curves due to the combined effect of high lateral and low vertical load on the outer wheel at low speed

On curves according to TSI, (2008) with a radius less than 290 m and according to GC/RT5021, (2009) with a radius less than 320 m, the cant shall be restricted to the limit given by the formula:

2-6

$$h_{t,\max} \leq \frac{(R-50)}{1.5}$$

Where: h_{max} is the cant in mm and R is the radius in m.

Exceptional limit of cant can be provided according to the judgment of engineer. It is undesirable to apply cant to the maximum value in exposed places of high winds experienced.

Table 2-4: Recommended and maximum limiting value of track cant for ERC (Yeserah, 2012)

Speed (km/hr)	Track Cant, h_t (mm)	
	Recommended Limiting Value	Maximum limiting Value
$200 \leq V \leq 300$	110	160
$160 \leq V < 200$	(110 – 150)*	160
< 160	150	160

Note: (110 – 150)* indicates that the lower value 110 for higher speed limit and high value 150 for lower speed limit. In addition the maximum value can be set at strict topography condition with special treatment of the track.

Equilibrium Cant

Equilibrium cant, h_{eq}, is the cant which gives a lateral acceleration of zero, a_y=0, for a given radius and vehicle speed.

2-7

$$h_{eq} = \frac{S}{g} * \frac{v^2}{R}$$

For the standard track gauge, S = 1500 mm, gravitational acceleration, g = 9.81 m/s2, radius in m and speed V (km/hr); therefore, the equilibrium cant in mm

2-8

$$h_{eq} = \frac{1500mm}{9.81m/s^2} * \frac{v^2}{3.6^2 * R} = 11.8 * \frac{v^2}{R}$$

Cant deficiency

When, passenger and freight trains run on the same track at different speeds, which means that ideal cant for the top speed would result in considerable excess cant for the slow-running traffic. A compromise is, therefore, to accept a certain degree of cant deficiency for the fast trains, producing flanging on the high rail and thus lateral wear of the rail head. Cant deficiency h_d is the difference between ideal cant (equilibrium cant) and actual cant and must satisfy the condition (Esveld, 2001);

2-9

$$h_d = h_{eq} - h_t = \frac{S}{g} * a_y$$

Where: a_y=lateral acceleration

Cant excess

Cant excess is introduced when the actual cant is higher than the equilibrium cant. It is the difference between actual cant and equilibrium cant and is defined as (Lindahl, 2001):

2-10

$$h_e = h_t - h_{eq}$$

It is achieved when the vehicle is running at a lower speed than the design speed of the track and cant excess should not be too high for slow trains. On high cant, the low wheels and rails would be highly loaded, possibly causing track deterioration. In particular this would in turn, high cant excess, leads excessive wear and damage on the low rail. Cant excess is related with lateral acceleration in the same way of cant deficiency.

2-11

$$h_e = -\frac{S}{g} * a_y \text{ ; When } a_y < 0, \text{ and } h_e > 0$$

2.2.2.4. Horizontal Curve Radius

The parameters that shall be considered in the determination of the minimum curve radius as cited by Lindahl (2001) are:

- The maximum and minimum operating speed;
- The applied cant; and
- The limiting values for cant deficiency and cant excess.

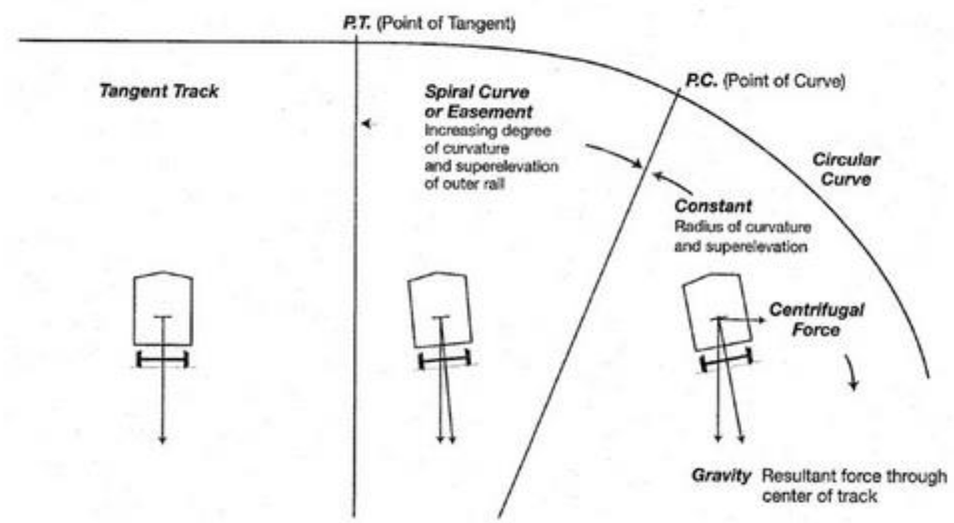


Figure 2-4: Horizontal curve elements

According to AREMA (2009) for standard gage track; horizontal curve radius is given by:

$$R = 11.8 * \frac{V_{\max}^{2}}{h_t + h_d} \tag{2-12}$$

$$R = 11.8 * \frac{V_{\min}^{2}}{h_t - h_e} \tag{2-13}$$

Where: V is in [km/hr]; h_{eq} in [mm]; and R is in [m]

2.2.2.5. Transition Curve

The following are the objectives of a transition curve; which is known with variable in radius.

a) To decrease the radius of the curvature gradually in a planned way from infinity at the straight line to the specified value of the radius of a circular curve in order to help the vehicle negotiate the curve smoothly

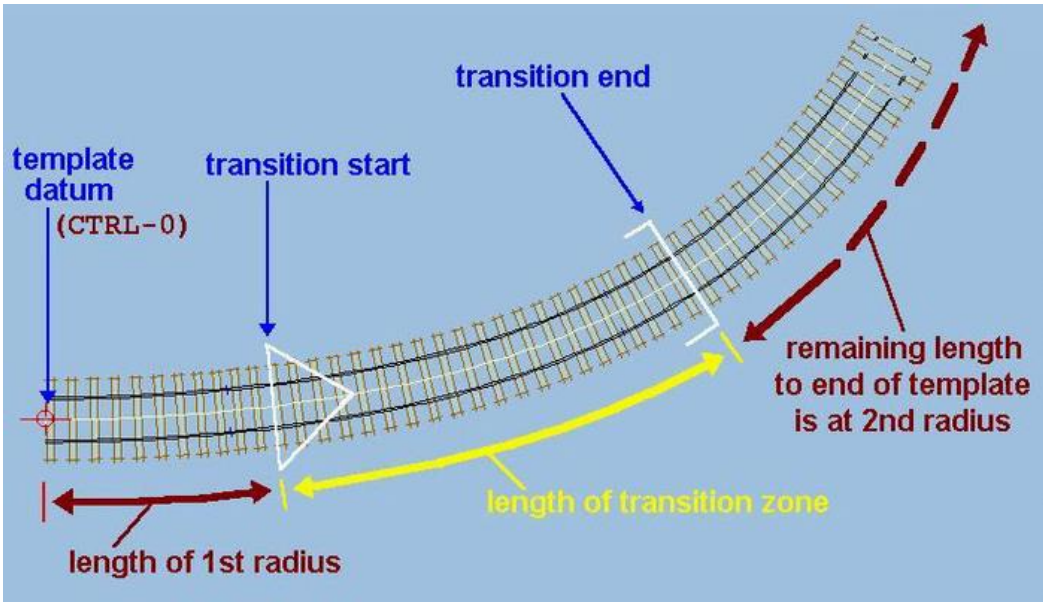


Figure 2-5: transition curve length

b) To provide a gradual increase of the superelevation starting from zero at the straight line to the desired superelevation at the circular curve.

c) To ensure a gradual increase or decrease of centrifugal forces so as to enable the vehicles to negotiate a curve smoothly.

The steepest permitted designed cant gradient shall be 1 in 400. Designed cant gradient value can be expressed in terms of length of transition curve/superelevation ramp and cant difference through this length as (Lindahl, 2001):

2-14

$$\frac{1}{n} = \frac{\Delta h_t}{1000 * L_t} \leq \frac{1}{400}$$

The length of the transition curve L_t the larger value can be selected from the following formula using CEN provisional standard as reviewed by (Yeserah, 2012):

2-15

$$L_t \geq \frac{V_{max}}{3.6} * \Delta h_d \left(\frac{dh_d}{dt}\right)^{-1}_{lim}$$

2-16

$$L_t \geq \Delta h_t \left(\frac{dh_t}{dx}\right)^{-1}_{lim}$$

Where:

- *Lt-length of transition curve in (m)*
- *Vmax- maximum line speed in (km/hr) with both variation of cant and variation of cant deficiency measured in (mm)*
- *(dht/dx)-1-inverse of cant gradient in (mm/mm)*
- *(dhd/dt)-inverse of rate of cant deficiency*

Example 2-3: *Compute transition curve length [m] for line of speed 180 km/h with both variation of cant and variation of cant deficiency of 100 mm. optimize rate of cant gradient (mm/s) of 30,40,50,60,70; and cant gradient (mm/mm) of 600,750, 1000,1200,1500.*

<u>Solution:</u>

Step-1: compute transition curve length with variation of cant deficiency of 100mm, and line speed of 180km/hr

$L_t \geq \frac{V_{max}}{3.6} * \Delta h_d \left(\frac{dh_d}{dt}\right)^{-1}$		
Δh_d (mm)	dh_d/dt (mm/s)	L_t (m)
100	30	167
	40	125
	50	100
	60	83
	70	71

Step-2: compute transition curve length with variation of cant of 100mm, and line speed of 180km/hr

$L_t \geq \Delta h_t \left(\frac{dh_t}{dx}\right)^{-1}$		
Δh_t(mm)	$(dh_t/dx)^{-1}$ (mm/mm)	L_t (m)
100	600	60
	750	75
	1000	100
	1200	120
	1500	150

Step-3: For specified variables the greater of the two computed is chosen as transition curve length.

Assume a rate of cant deficiency of 40 mm/s, and then the transition length computed using this value is 125 m. If the cant gradient is steeper than (greater than) 0.8mm/m (less than $(dht/dx)^{-1}$ = 1250 mm/mm), the computed transition curve length become less than 125 m. Therefore, for the given assumption, the first equation which gives greater value (governing value) can be selected.

2.2.2.6. Gradient

The topographical conditions usually require some kind of longitudinal gradients, along the way.

a) When a train climbs a certain upgrade at a uniform speed and passes over the summit of the curve, an acceleration begins to act upon it and makes the trains to move faster and increases the draw bar pull behind each vehicle, causing a variation in the tension in the couplings
b) When a train passes over sag, the front of the train ascends an up-grade while rear vehicles tend to compress the couplings and buffers, and when the whole train has passed the sag, the couplings are again in tension causing a jerk.

The following types of gradients are used on world railways

I. **Ruling Gradient:** The steepest gradient that exists in a section. It determines the maximum load that can be hauled by a locomotive on that section. All other gradients provided in a given section should be done flatter than the ruling gradient.

II. **Pusher or Helper Gradient:** In hilly areas, sometimes, gradients steeper than the ruling gradient are provided to reduce the overall cost. In such situations, one locomotive is not adequate to pull the entire load, and an extra locomotive is required.

III. **Momentum Gradient:** Steeper than the ruling gradient. In valleys, a falling gradient is sometimes followed by a rising gradient. In such a situation, a train coming down a falling gradient acquires good speed and momentum; this gives additional kinetic energy to the train and allows it to negotiate gradients steeper than the ruling.

IV. **Gradients in Station Yards:** The gradients in station yards are quite flat to prevent standing vehicles from rolling and moving away from the yard due to the combined effect of gravity and strong winds. Yards are not leveled completely and certain flat gradients are provided in order to ensure good drainage.

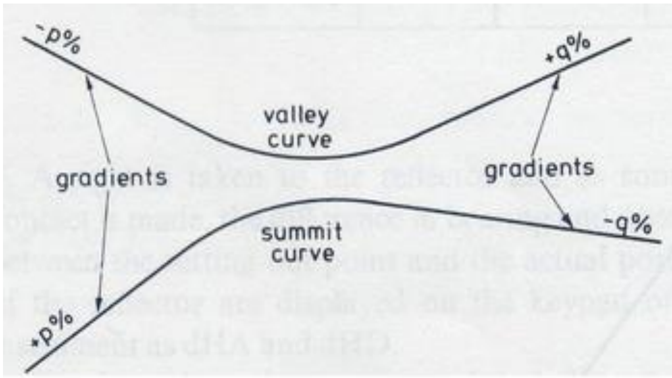


Figure 2-6: Sags and summits of vertical curve

Design of track gradients shall take into account of the following factors:-

a) Braking and traction performance of vehicles likely to use the line;
b) Position of signals and operational regime (for example, the likelihood of a train being required to start on the gradient or stop at a station or signal);
c) Projected rail adhesion conditions, including the effect of the weather; and
d) The combined effect of gradient and horizontal curvature where the gradient coincides with a small radius horizontal curve

Generally a recommended limit of gradient is 1% in normal condition. But depending on the length of continuous gradient, type of traffic lines (passenger or freight) and braking capacity of a train, value of gradient higher than 1.25% can be provided even for mixed traffic lines.

According to TSI recommendation [TSI, 2008], in consideration of: High traction power; reducing length of continuous gradient;

a) The slope of the moving average profile over 3 km is less than or equal to 2.0% and
b) The maximum length of continuous 3.5 % gradient does not exceed 0.5 km.

2.2.2.7. Vertical Curve radius

Possible vertical curve radius with minimum values according to TSI (TSI are preferred since it is formulated for design speed of less than 230 km/h) can be recommended by the following equation (Esveld, 2001)

$$R_v = \frac{V_{\min}{}^2}{12.96 * a_v} \geq R_{v,\lim}$$

Where: a_v =0.22 m/s^2 and 0.31 m/s^2 (vertical acceleration) recommended and maximum limiting values respectively for mixed traffic line of speed less than or equal to 230 km/hr.

There are different minimum values of vertical curve radius dependent of different requirements of limiting values on a crest or in hallow. Normally, the required vertical radius is somewhat larger on crests than in hallows. This is due to the risk of wheel unloading on crests.

NOTE: The minimum and maximum limiting values of the geometric elements are dependent on compulsory standards of nations and other standards for which countries are a member.

Example 2-4: *Calculate the superelevation and maximum permissible speed for a 2" SG transitioned curve on a high-speed route with a maximum sanctioned speed of 110 kmph. The speed for calculating the equilibrium superelevation as decided by the chief engineer is 80 kmph and the booked speed of goods trains is 50 kmph.*

<u>Solution:</u>

Step-1: radius of curve

$$R = \frac{1746}{\alpha} = \frac{1746}{2} = 873m$$

Step-2: Superelevation for equilibrium speed

$$= 11.8 * \frac{v^2}{R} = 11.8 * \frac{80^2}{873} = 86.5mm$$

Step-3: Superelevation for maximum sanctioned speed

$$= 11.8 * \frac{v_{\max}{}^2}{R} = 11.8 * \frac{110^2}{873} = 163.55mm$$

$$cant\ deficiency = 163.55 - 86.5 = 77.05mm$$

Step-4: Superelevation for goods trains with a booked speed

$$= 11.8 * \frac{v_{\min}{}^{2}}{R} = 11.8 * \frac{50^{2}}{873} = 32.62mm$$

$$cant\ excess = 86.5 - 32.62 = 53.88mm$$

Step-5: Maximum speed potential or safe speed of the curve as per theoretical considerations, being a high-speed route:

$$R = 11.8 * \frac{V_{\max}{}^{2}}{h_t + h_d} ; 873 = 11.8 * \frac{V_{\max}{}^{2}}{86.55 + 77.05}$$

$$V_{\max} = 110.1kmph$$

Step-6: The maximum permissible speed on the curve is the least of the following:

- Maximum sanctioned speed, i.e., 110 kmph
- Maximum or safe speed over the curve based on theoretical considerations, i.e., 110.1 kmph
- There is no speed constraint due to the transition length of the curve Therefore, the maximum permissible speed over the curve is 110 kmph and the superelevation to be provided is 100.8 mm or approx. 100 mm.

2.2.3. Subgrade

Subgrade is the naturally occurring soil which is prepared to receive the ballast. During the operation time the loads comes from the upper layer need to have a stable foundation to protect them from different problems. Due to this reason the subgrade has a great advantage in the track performance and maintenance; it limits progressive settlement from repeated traffic loading; limit consolidation settlement; prevent massive slope failure; and restrict swelling or shrinking from water content change.

If the subgrade soil cannot achieve its required function, a capping layer of granular material, called formation, is placed. The formation can be in the shape of an embankment or a cutting. When formation is in the shape of a raised bank constructed above the natural ground, it is called an embankment. The formation at a level below the natural ground is called a cutting.

- The height of the formation depends upon the ground contours and the gradients adopted.
- The side slope of the embankment depends upon the shearing strength of the soil and its angle of repose.
- The width of the formation depends upon the number of tracks to be laid, the gauge, and such other factors.

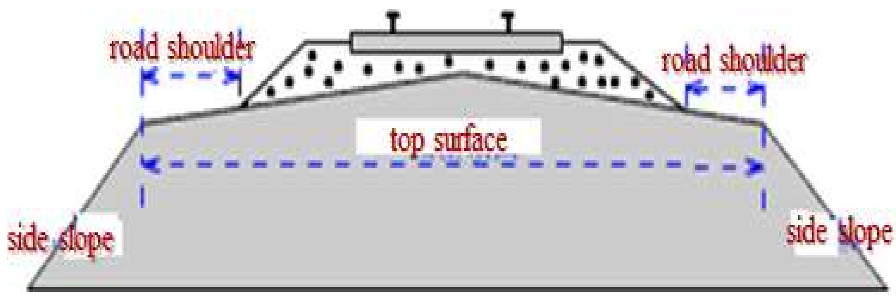


Figure 2-7: section of subgrade

The four basic geometric features are:

- Width of top of subgrade or bottom of cut
- Height of fill or depth of cut
- Side slopes of fill or cut
- Provision for drainage

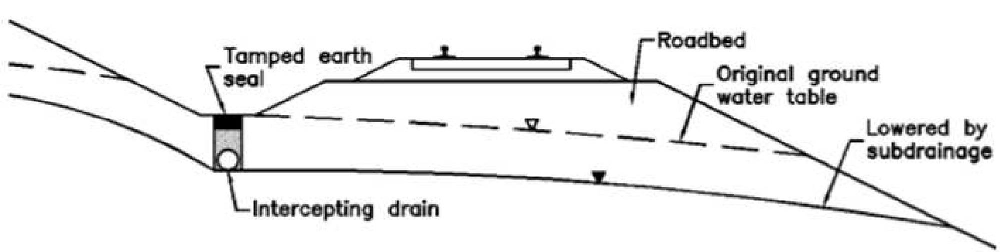


Figure 2-8: Interception of sidehill seepage by subdrainage

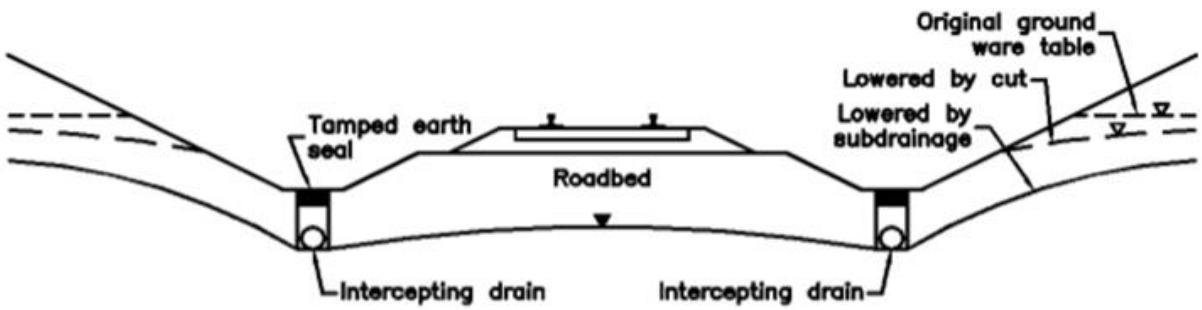


Figure 2-9: lowering of ground water in a wet cut

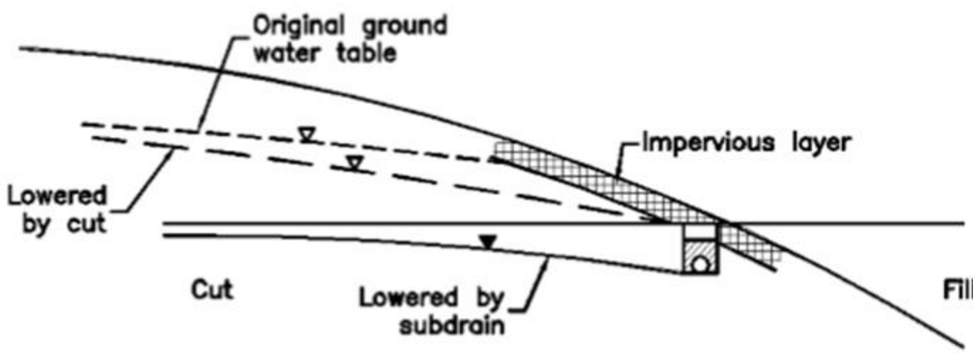


Figure 2-10: lowering of ground water in cut to fill transition (longitudinal)

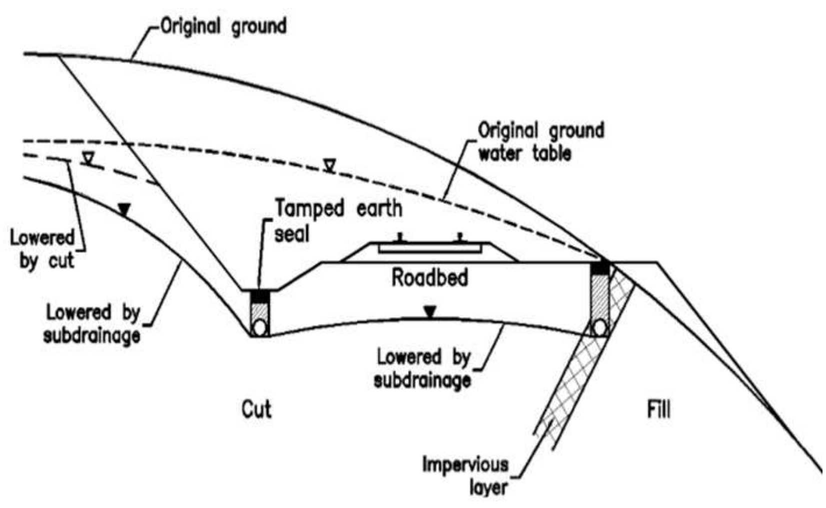


Figure 2-11: lowering of ground water in cut to fill transition (sidehill)

2.2.3.1. Subgrade Failure

Subgrade has a major influence on track performance. Three kinds of subgrade failure associated with train loading can be distinguished based on mechanism of failure.

a. Massive shear failure

The driving forces are the weights from the train, the track superstructure and the unbalanced section of substructure. The resisting force is from the substructure layer shearing resistance. Because more of the subgrade failure zone is in the subgrade, then the subgrade strength properties have a big effect on the factor of safety against massive shear failure.

Cut slope stability problems often arise because of ground water seepage into the excavated slopes. Other problems are weathering of freshly exposed soil and rock, and volume changes in expansive clays.

b. Progressive shear failure

Stresses imposed on the subgrade by the axle loads may be large enough to cause progressive shear failure (general subgrade failure). This condition will most likely develop in the top part of the subgrade where the traffic induced stresses are highest. Overstressed soil will be squeezed sideways from beneath the track and upwards to give this bearing capacity failure.

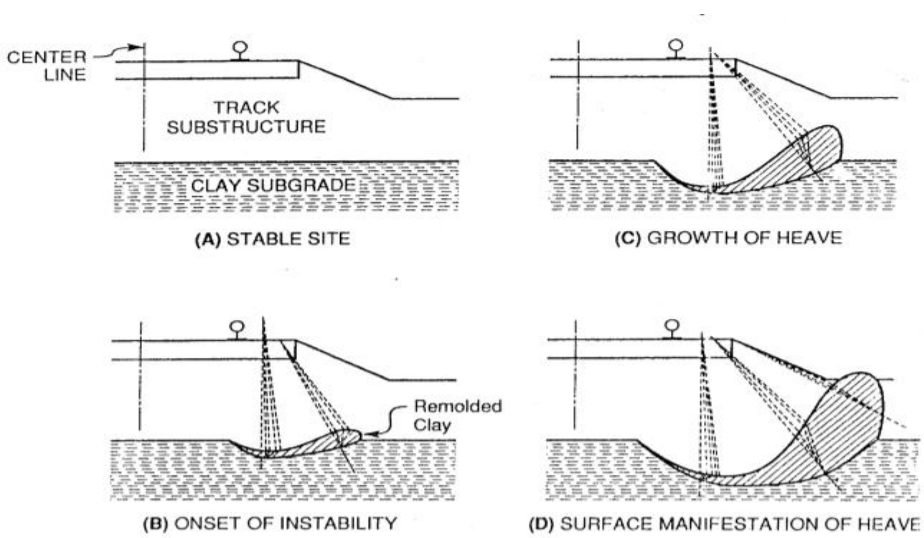


Figure 2-12: movement of overstressed clay

The probability of progressive shear failure can be minimized by:

- Ensuring that an adequate depth of distributing (granular) material exists between the underside of the sleeper and the surface of the subgrade.
- Ensuring that the drainage system maintains a low water table level

The better solution to the problem could well be the removal of the ballast layer, excavation of the subgrade to reduced elevation, and replacing ballast and subballast up to its original level. The resulting increase in granular depth will reduce the intensity of stress applied to the surface of the subgrade.

c. Attrition

Attrition (local subgrade failure) of the subgrade by the overlying ballast in the presense of water can result in the formation of slurry at the ballast/sugrade interface. Under certain conditions, cyclic loading associated with passing traffic can cause this slurry to be pumped up to the surface of the ballast. Such failures are normally associated with hared fine grained materials clay, and soft rocks, such as chalk.

2.2.3.2. Physical Properties of Subgrade Soil

The classification of soil according to AASHTO and USCS are based on the grain size distribution and Atterberg limit test of the soil.

I. ***Grain Size Distribution:-*** It is one of the most important soil characterizations as the particle size distribution affects many properties of the soil such as density, strength, void ratio, and permeability

II. ***Atterberg limits***: describes the consistency and plasticity of fine-grained soils with varying degrees of moisture content for the portion of the soil passing the No. 40(0.425 mm) sieve. The method classifies soil into liquid, plastic, semi-solid and solid state.

2.2.3.3. Engineering Properties of Subgrade Soils

Two engineering properties of soils are important to many types of engineering works.

I. Compaction (Moisture-Density Relation Test)

Compaction tests are performed using disturbed, prepared soils with or without additives on soil passing the No. 4 (4.75mm) or 19mm sieve. The main outcomes being sought are:

- Increase shear strength and therefore bearing capacity;
- Decrease void ratio and therefore reduce future settlement and permeability; and
- Decrease undesirable volume changes such as swelling and shrinkage

II. Strength and stiffness properties

The strength and stiffness of materials is largely related to their compaction density, which depends significantly on the particle size distribution

CBR- value: is used as an index of soil strength and bearing capacity which is the main subgrade parameter commonly used in design of track. Practically it is not possible to build upon subgrades whose CBR value of the soil is less than 3

Resilient Modulus (MR): It is a subgrade material stiffness test and actually it is an estimate of modulus of elasticity (E). The minimum value of modulus of elasticity, for lines to be used by high-speed trains or where high loads are planned, is 120 MPa under permanent soil moistness.

2.2.3.4. Subgrade Soil Stabilization

Stabilization of subgrade is done for some soils when compaction will not produce the desired strength needed to support structures. The two frequently used methods of stabilizing soils are stabilization by mechanically or stabilization by chemical additives.

Stabilizer selection should be based on the effectiveness of a given stabilizer to improve the physiochemical properties of the selected soil.

2.2.3.5. General Design Requirements for Earth Structures

The design for the earth structures provided by the Designer in the preliminary stage, mostly include ways to meet the environmental constraints, to anticipate all possible difficulties, to achieve balanced cut/fill volumes, and to produce optimized structures from the technical, financial, time and environmental point of view. From the perspectives listed above; the following tasks, but not limited to, are expected to be done:

- Evaluate the probable ground condition by collecting all possible information on ground investigations already realized on locations close to the considered site and from construction of similar earth structures close to the project
- Define the geometry of the structure and right-of-way
- Assess the cost and time of execution of works,
- Ensure the reliability of the work
- Achieve well-balanced cut/fill volumes
- Assess the potential risks during construction and in operation
- Define performance targets for construction sufficiently explicit so as to allow recognition of the conformity
- Limit the costs of maintenance during operations.
- Maximize technical and financial considerations for the structure at each phase: design, construction and maintenance during operation

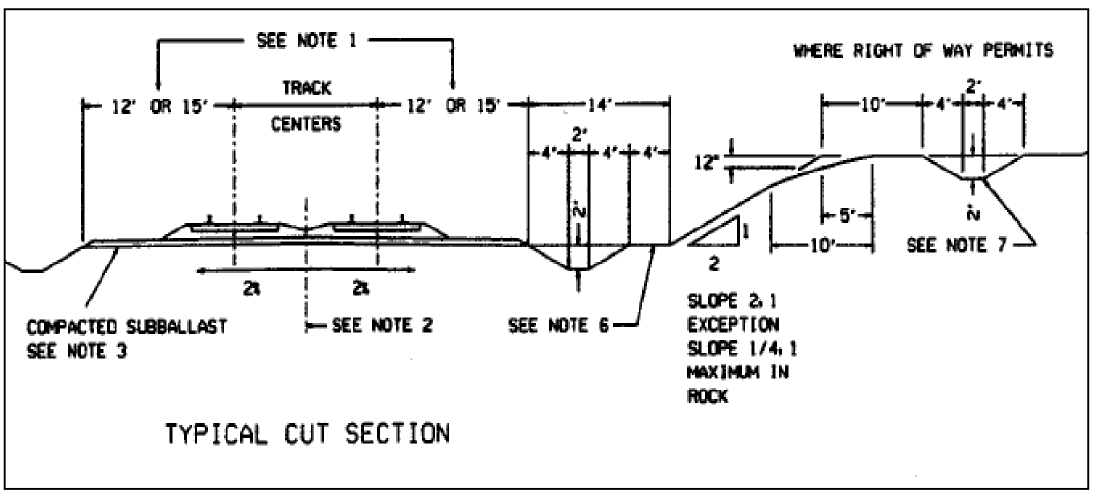


Figure 2-13: Typical road bed section

2.2.4. Capital Cost Estimation

Capital cost estimates should cover:

i. **land acquisition:-** account for major proportion of project development cost in urban areas (especially in case of urban mass transit railways)
ii. **civil works:-** account for major share of development cost of projects involving extensive tunneling and bridge work (around 50% in case of Chinese High Speed lines)
iii. **signaling and power and control system installation; and initial rolling stock acquisition:-** comprise up to 25% of High Speed development costs (e.g. case of Chinese 350km/hour lines)
iv. track-laying

2.2.5. O&M Cost Estimates

As railway systems became more complex and as funds, public or private, became scarcer, costs, such as operational and maintenance costs, became important considerations.

Considerations of the following questions give rise to the subject of Lifetime Cost (LTC).

- How long should we be concerned about the operation and maintenance costs? When do we want to replace or retire the system?
- What is the system's life span?
- What is the intended life, i.e., how long do we plan to keep the equipment or system in service? And,
- What is the estimated technical life, i.e., how long will the system stay operational before its technical characteristics fall below an acceptable level?

Relevant O&M costs include costs of: train crews, fuel or electric energy consumption, locomotive and rolling stock maintenance, infrastructure maintenance (fixed and variable elements), station and train control staff

2.2.6. Financial Analysis

It comprises a detailed Discounted Cash Flow (DCF) analysis that will determine the project's financial viability under alternative assumptions with respect to:

i. revenue generation;
ii. project capital and O&M costs; and
iii. project financing arrangements

Positive cash flows of railway will mostly comprise revenue from collection of passenger fares, freight tariffs, or both–in case of High Speed and Urban Transit lines; additional revenue may be collected from retail concessionaires in stations. **Negative cash flows of railway** will mostly comprise its capital cost, financing cost, and Operating and Maintenance (O&M) costs.

Project will usually be financed by combination of different sources: Equity funds, long term foreign government or international agency loans, commercial loans, or in the case of high profile projects (like High Speed lines) from bond issues.

2.2.7. Economic Analysis

It involves estimation of the net benefits of the project to society through a comparison, over its life, of its economic benefits with its economic, or shadow-priced, capital and operating costs; analysis of economic net benefit flows (benefits less costs), including calculation of EIRR and B/C indicators. Well-designed railway projects will usually achieve significant EIRR's (in the range of 20-30%) –well in excess of the long term cost of capital. Shadow pricing involves the removal of taxes and government charges from costs, as well as compensation for price distortions (where prices not determined by market forces)

Economic benefits from different types of railway projects

- High Speed Passenger Railway will offer substantial time savings compared with road and air (when all access costs are considered),
- whereas freight railway will not offer time savings but by diverting traffic from road will save road operating costs (as its major economic benefit)
- New traffic generation and Value of Time Savings likely to be highest for High Speed and Urban Mass Transit railway

- Development of High Speed and Urban Mass Transit railway will also have greatest impact in terms of raising land values and of employment generation along railway routes
- Freight railways will offer highest reductions in road operating costs, road accident costs and road maintenance costs, owing to diversion of traffic away from heavy trucks on national highways.

Example 2-5: *Financial and economic indicators can be calculated like the following in excel sheet*

	A	B	C	D	E	F	G
1	**MARR**	**12%**					
2	***Year***	***Operation year***	***Total cost, C (USD)***	***Total benefit,B (USD)***	***IBT USD, B-C***	***Tax USD, 30%*(B-C)***	***IAT USD, (B-C-Tax)***
3	1						
4	2						
5	.						
6	.						
7	.						
8	.						
9	n						
10							
11	***FNPV***	For exel use=NPV(B1,G3:G9)					
12	***ENPV***	For exel use=NPV(B1,E3:E9)					
13	***FIRR***	For exel use=IRR(G3:G9)					
14	***EIRR***	For exel use=IRR(E3:E9)					
15	***B/C***	for exel use=NPV(D3:D9)/NPV(C3:C9)					fantish

Sheet1

2.2.8. Sensitivity Analysis

It analyze how the optimal solution is affected by changes, within specified ranges, in the values of the project economic parameters

Sensitivity analysis is conducted to get specific information out of the model. Sensitivity analysis is done by selecting an important factor, and changing the value of this factor while the other factors have fixed values. This way it is possible to see which factors have a big impact on the overall performance of the simulation mode

Example 2-6: *Let $EIRR_o$ is the optimal economic internal rate of return of a given railway project X, associated with base-case scenario, sensitivity analysis can be done in Excel sheet like the following;*

Step-1: fill the blank cells in the table with corresponding EIRR values

	Economic factors		
Increments	*Initial investment cost*	*Operational cost*	*Annual income*
10% increase			
5% increase			
Base-case	$EIRR_o$	$EIRR_o$	$EIRR_o$
5% decrease			
10% decrease			

Step-2: graph economic factors versus increments

- The project will be most sensitive to the economic factor with steepest slope of curve.

EXERSISE 2-1:

1. *The total number of trips from zone* ***A*** *to zone* ***B*** *is 4200. Currently all trips are made by car. Government has two alternatives- to introduce a train or a bus. The travel characteristics and respective coefficients are given in table below. Decide the best alternative in terms of trips carried.*

	t^v_{ij}	t^{walk}_{ij}	t^t_{ij}	F_{ij}	ϕ_{ij}
coefficient	0.05	0.04	0.07	0.2	0.2
car	25	-	-	22	6
bus	35	8	6	8	-
train	17	14	5	6	-

2. *According to USID (2011), in its report on greenhouse gas emission in east Africa, the energy sector is Ethiopia's and Tanzania's third highest source of GHG emissions.*

Ethiopia, Tanzania's and Kenya's energy emissions combined are responsible for 87% of the regions total energy sector GHG emissions. According to the WHO, in 2013 the road crash fatality rate in Ethiopia was 4984.3 deaths per 100,000 vehicles per year, compared to 574 across sub-Saharan African countries. It is known that developing countries should increasingly turn to regional integration in response to the challenge of globalization;

The Government of Ethiopia has been looking forward to resume diplomatic ties, transport, trade, and communication links with Eretria after a bilateral peace summit and agreement took place on 8-9 July 2018 in Asmara. Let the two governments initiate a study for the development of additional railway network connecting Shire-Asmara-Massawa which is an extension of route-5 of ENRP.

a. Show a methodology how you would conduct social and environmental impact assessment for railway network connecting Shire-Asmara-Massawa.
b. Suggest alternative scenarios, as a methodology, to be followed during construction, maintenance, and operation that mitigates impact of the project on the environment?
c. Which specific elements and how you would investigate for stable earthwork design; during a preliminary stage?
d. How you will optimize factors such as: shortest route with steep grades; or longest route with gentle slope; to choose the better route based upon overall construction, operational and maintenance criteria?
e. What is your insight on political interventions during route selection; towards proper railway line planning?
f. Select a better combination from financing options below, for the above case-example, and why?
    - Equity funds,
    - Long term foreign government or international agency loans,
    - Commercial loans,
    - bond issues

3. Which special safety considerations, in urban areas, will you take for the construction of railroad sections fitted for elevated, at grade and below grade? And why?

MODULE-THREE

3. RAILWAY TRACK STRUCTURE

Introduction

Track is defined as an assemblage of rails, ties and fastenings over which cars, locomotives and trains are moved. Ballast may or may not be a part of the track, depending on the type of the track in question. The main function of the railway track structure in the railway transportation is to provide safe, smooth, economical and comfortable railway transportation for the society.

The main types of rail track for urban and regional rail applications are the following.

I. **Ballasted track:** are, a traditional railway track form, mainly used for regional transport of passengers and goods. It consists of rails and sleepers mounted on a ballast bed.

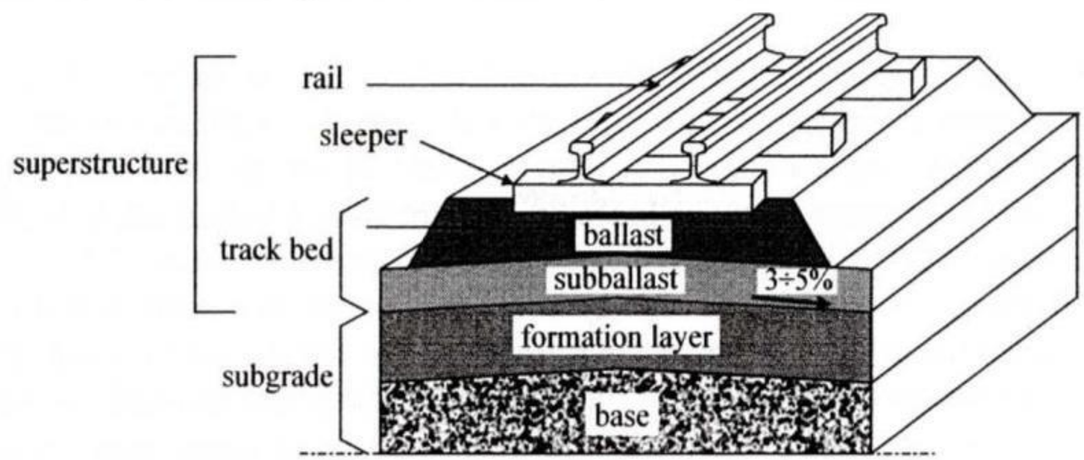


Figure 3-1: Typical ballasted track

II. **Covered track:** are mainly used in cities (trams etc.) where space is limited and road or pedestrian traffic may also need to use the area where the track is installed. Only the rails are exposed on the surface and the other infrastructure is covered by a road surface.

Figure 3-2: Nottingham Express Transit (NET) - Tram Track

III. **Slab track:** are mainly used for high-speed rail tracks, tracks in tunnels, tracks on bridges and tracks which require little maintenance (e.g. covered tracks and green tracks). It consists of rails and/or sleepers mounted or cast into a solid base.

Figure 3-3: Fiber Reinforced Concrete Slab Track

Table 3-1: advantage and disadvantage of track type

Track Type	Advantages	Disadvantages
Ballasted	Low construction costs and inherent vibration damping	Ballast may degrade and need replacement or realignment i.e. costly and time consuming
Covered	Track areas can be used by other modes of transport	Replacement is costly, timely and disruptive to other traffic since the track is embedded in the road surface
Slab	Maintenance free	Costly and difficult to adapt/change

Many recent advances in ballasted track construction make them even more competitive against slab track. Few examples of these advances are the following:

- Introduction of reinforced concrete or steel sleepers.
- Optimization of the rail geometry improves the wheel-rail contact condition offering a better load bearing capacity.
- Optimized rail pads and fastening providing better attenuation of the wheel-rail contact forces and of noise.
- Stone blowing and tamping machines provide more accurate corrections and faster maintenance.
- Better track resilience and resistance to settlements through the use of under sleeper mats as well as geo-grids and geo-synthetics between the different layers.
- The use of geotechnical retro-fitting techniques (e.g. lime cement columns), improving the bearing capacity of the subgrade, minimizing the possibility of settlements.

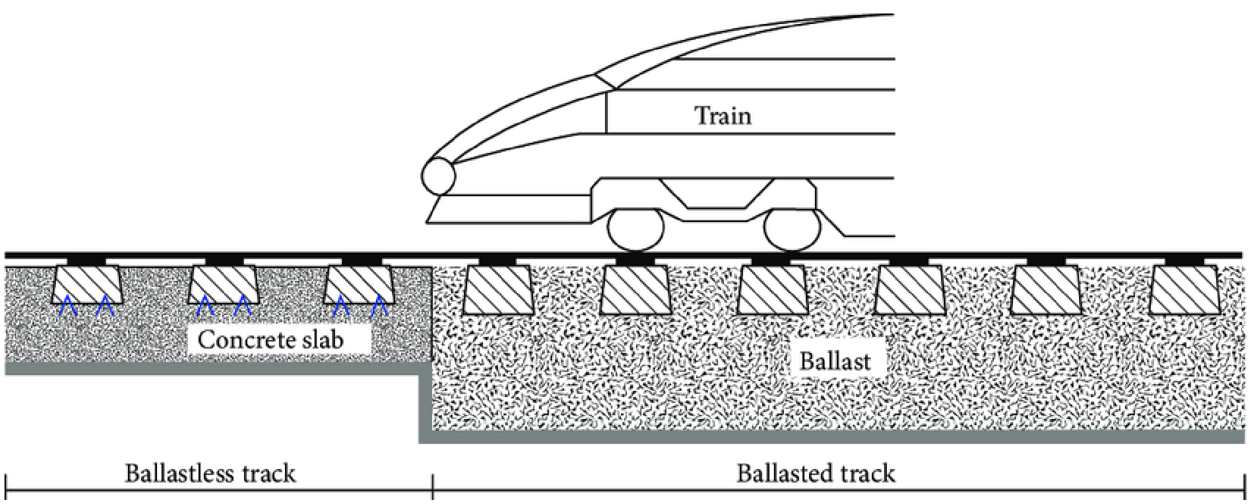


Figure 3-4: ballasted vs. ballastless track

The ballast and subballast layer in granular trackbed can be replaced with a HMA layer in HMA trackbed for slab track; and capping layer can be provided to protect the natural ground or fill from moisture ingress and to form a unified subgrade layer.

Concrete is the dominant material in slab track application all over the world and only in very special occasions asphalt has been used as materials for slab track construction, and this is as a result of high construction demands.

3.1. Rail

Rails are made of steel which constructed longitudinally for the first element in contact with the vehicle wheels. Its main function is for the transmission of the imposed loads that come from the wheel.

The rail sections are connected in the field by either bolted joints or welding. On today's high performance lines, rails are welded together to create continuously welded rails (CWR) thus eliminating a potential cause of dynamic load: the discontinuity caused by traditional fishplate bolted joints.

3.1.1. Properties and Profiles of Rail

Properties of rail

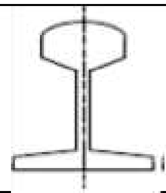
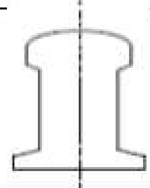
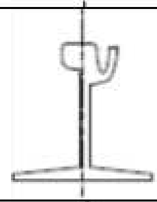
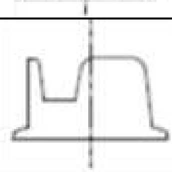
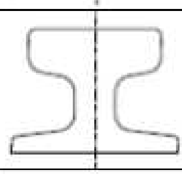
Rail properties to be considered when choosing the size and type of rail are: wearability, hardness, ductility, manufactures defects in the rail material and Rail straightness.

Rail profile

To provide flexural stiffness and strength, rail is shaped in section somewhat like an I-beam. But the head is made narrower and deeper than the flange of an ordinary I-beam to resist the contact pressure and wear from flanged wheels better.

The most commonly used rail profile is flat-bottom rail, also called Vignole rail, and is divided into three parts: rail head, rail web and rail foot.

Table 3-2: Types of rail profiles and their applications (Esveld, 2001)

Shape	Profile type:	Applications
	Flat-bottom rail	Standard rail track
	Construction rail	Manufacturing of automobiles and switch parts
	Grooved rail	Railway track embedded in pavements, roads, yards
	Block rail	Railway track used in concrete slab as part of Nikex-structure
	Crane rail	Heavy load hoisting cranes with high wheel loads

Steel Rail Grades

The mass per metre of rail is known to contribute to the track stability.

Table 3-3: Chemical composition and tensile properties of AREMA standard rail steels (AREMA, 2009)

C (%)		Mn (%)		Si (%)		Standard Rail			High Strength rail		
Min	Max	Min	Max	Min	Max	HB	f_{yk}	f_{yd}	HB	f_{yk}	f_{yd}
0.74	0.86	0.75	1.25	0.1	0.6	310	983	511	370	1180	828
Note: The maximum composition of Cr, Mo, Ni and V are 0.3, 0.06, 0.25 and 0.01 in percent. Chemical composition of low alloy steel rail is lower than this provision but Cr has a minimum of 0.25 and maximum of 0.4%. f_{yk} and f_{yd} are minimum tensile and yield strength of steel rail in MPa HB is Brinell hardness number in the head area only											

The Grade 700, with about 0.5%C, has a microstructure of about 30% ferrite and 70% pearlite within the rail head. Ferrite structure is soft which imparts ductility but is prone to wear. Increasing the carbon content leads to achieve a 100% pearlitic microstructure.

The wear resistant rails of Grade 900, which became the standard rail for main lines, have a coarse pearlitic microstructure with sufficient ductility and toughness for general application. The further strengthening of pearlitic rails to 1100-1200MPa tensile strength, with high wear resistant, is based on increased pearlite refinement.

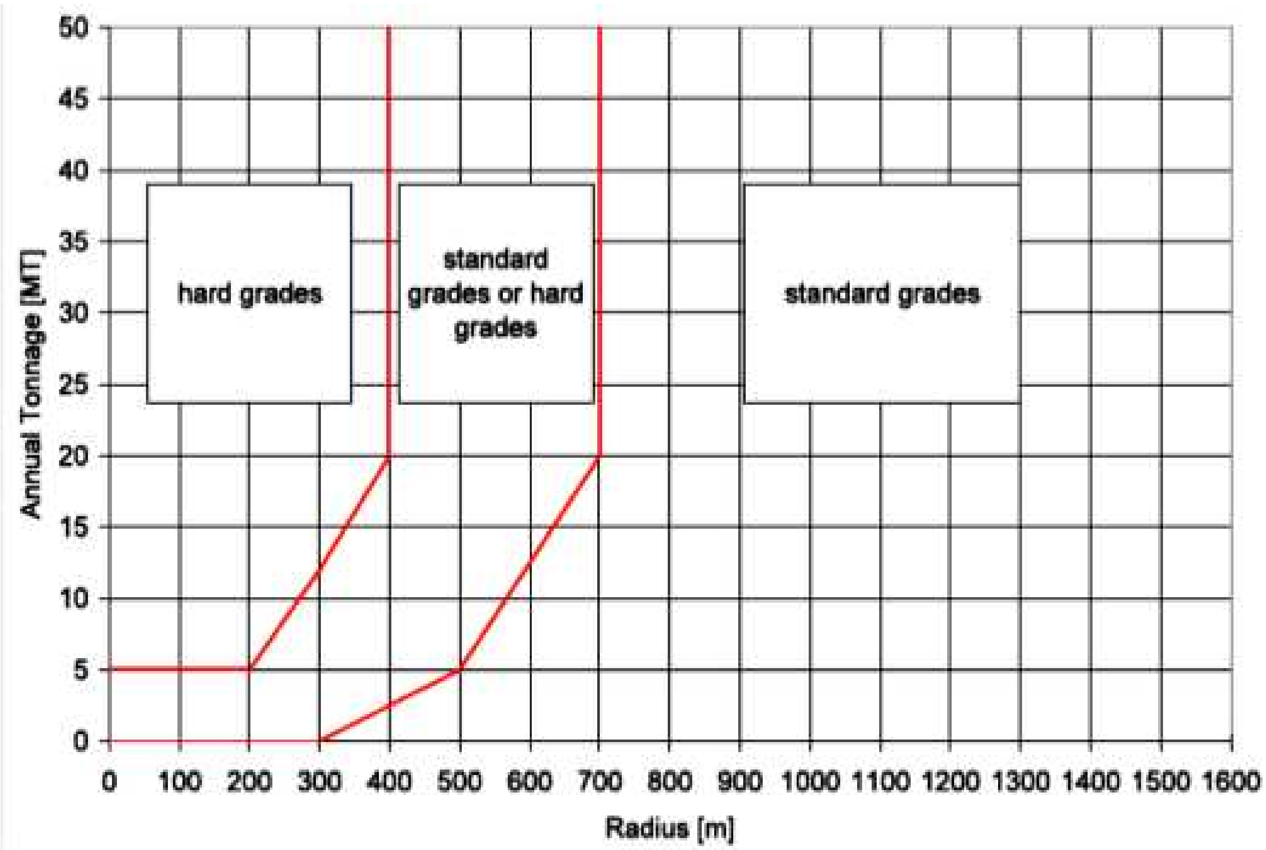


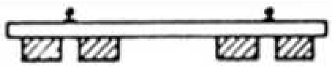
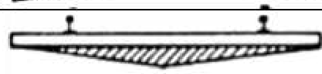
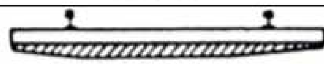
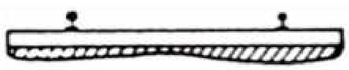
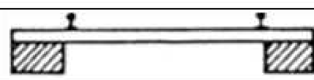
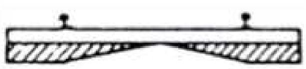
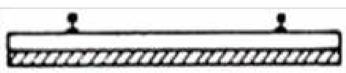
Figure 3-5: standard and hard steel grades according to UIC leaflet 721 (Pointner P., A. Joerg and J. Jaiswal, 2010)

3.2. Sleepers

In a ballasted track the rail rest on sleepers and together form the built up portion of the superstructure. The sleepers are often referred to as ties as they tie the rails together, preventing any dangerous relative lateral movement and providing support for the rails.

The main functions of sleepers are to distribute the wheel loads transferred by the rails through the rail seat to the bottom of the ties to provide an acceptable level or stress for the ties and ballast.

Table 3-4: Hypothetical distribution of sleeper bearing pressure (current practices) (Sadeghi J. M. and A. Babaee, 2006)

Distribution of bearing pressure	Remarks
	Laboratory test
	Tamped either side of rail
	Principal bearing on rails
	Maximum intensity at the ends
	Maximum intensity in the middle
	Center bound
	Flexure of sleeper produces variations form
	Well tamped sides
	Stabilized rail seat and Sides
	Uniform pressure

3.2.1. Sleeper Types

The sleepers mostly used on world Railways are classified according to:

A. **production material:** wooden sleepers, cast iron (CI) sleepers, steel sleepers, and concrete sleepers

B. **Usage:** regular sleeper, switch sleeper and bridge sleeper.

Table 3-5: advantage and disadvantage of sleeper types

Timber sleepers	
Advantage	Disadvantage
- good resilience, - ease of handling, - adaptability to nonstandard situations or - electrical insulation	- Durability limitations.
Steel Sleepers	
Advantage	Disadvantage
- long service life, - great dimensional accuracy and - positive residual value	- insulation, - maintenance using tampers and - Relatively high price.
Concrete sleepers	
Advantage	Disadvantage
- stable connection of CWR track due to its heavy load (200-300 kg); - freedom of design and construction - relatively simple to manufacture - Used to provide a cant to the rails to help develop proper rail wheel contact.	- Durability limitations. - less elastic than wood; - susceptible to corrugations and poor quality welds; - risk of damage from impacts (e.g. derailment); - dynamic loads and ballast stresses can be as much as 25% higher - Residual value is negative.

Concrete sleepers can be either two-block or mono-block. Monoblock sleeper has the shape of a beam and has roughly the same dimensions as a timber sleeper whereas two-block sleepers consist of two blocks of reinforced concrete connected by a coupling rod or pipe.

a) Mono-block concrete sleeper

b) Twin- bock concrete sleeper

Figure 3-6: types of concrete sleeper

The following parameters could be used to select the preferred sleeper material: life-cycle costs, sleeper design methodology, and Rail fastening systems.The choices of concrete sleepers, more widely used in the world, are because of different reasons; basically it is according to economic advantages. In case of Ethiopia, steel is relatively highly costly; and the concrete sleepers are not affected very much by either climate or weather, made concrete sleeper preferable than others.

Pre-tensioning is widely used in concrete railway sleepers. The principle behind prestressed concrete is that compressive stress induced by high-strength steel tendons in a concrete member before loads are applied will balance the tensile stresses imposed in the member during service.

Table 3-6: advantage and disadvantage of PSC

Prestressed concrete (PSC)	
Advantage	Disadvantage
Thinner and lighter than RCC sections Unlike RCC, whole concrete area is effective in resisting loads Show less deflection Good to withstand vibrations, impact, and shock	Requires very good quality control and supervision High tensile steel is, used in PSC, is about three times costlier than mild steel More brittle because of high tension steel

3.2.2. Spacing and Dimensions of Sleepers

Tie spacing

Tie spacing affects rail flexural stress, compressive stress on ballast and roadbed, lateral resistance of the track structure and the flexural stress in the ties themselves. The consequences of increasing tie spacing are higher rail bending moments and corresponding deflections and stresses within the individual ties; and It can reduce the fatigue life of the rail even though the stress levels are within allowable limits.

AREMA recommends a c/c spacing's of ties between 20 in. (510 mm) and 30 in. (760 mm) intended for track designs; where as In Australia standards a c/c spacing's of 500mm to 750 mm prestressed concrete sleepers are intended for track designs.

Dimensions of Sleepers

Use of longer, wider, or stiffer sleepers increases the sleeper-to-ballast bearing. There are, however, a limit beyond which an increase in sleeper size is ineffectual in reducing track stress and increasing track modulus Sleepers shall be designed to a preferred length of 2.5 m and a maximum depth at the rail seat of 250 mm.

AREMA, (2009) recommends: between 7 ft. 9 inches (236 cm) and 9 ft. (274 cm) in length and between 8 inches (20 cm) and 13 inches (33 cm) in width at their bottom surface.Depth of any section of tie shall not be less than 6 inches (150 mm) and shall not be exceed 10 inches (250 mm) respectively.

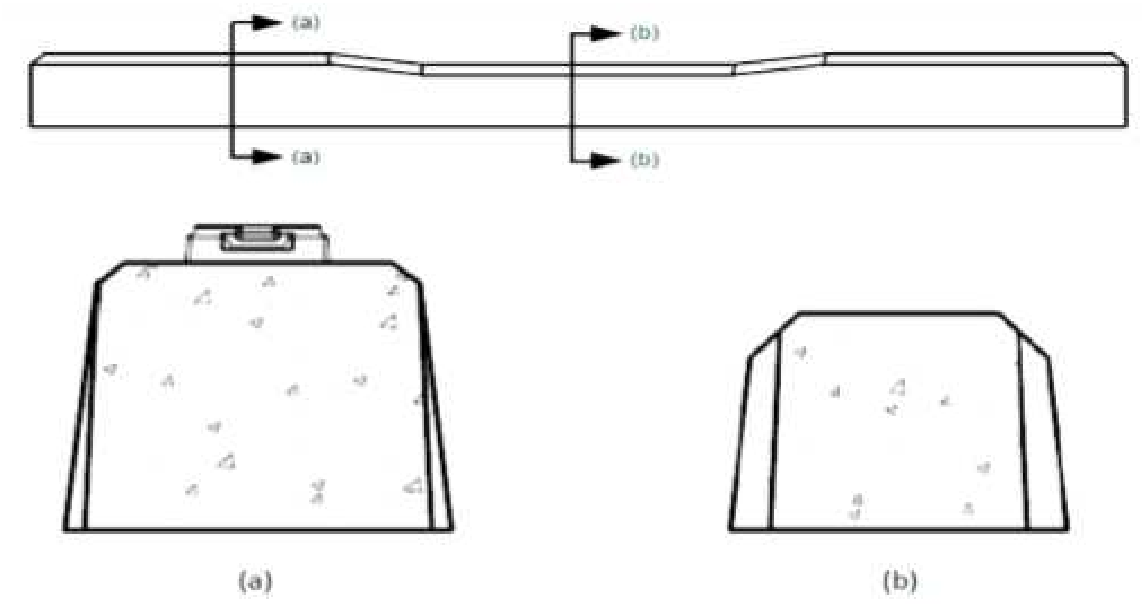


Figure 3-7: Typical tie cross section profile (a) rail seat section (b) center section (Lutch, 2009)

3.3. Fastening System

The fastening system or "fastening" includes any device or system of components used to fasten the rail to the tie or other support. The primary components of fastening systems are fastener and rail pad. Some tracks might have base plates with or without pads, which helps the workmen to remove damaged rails without having to untie the fastenings and immediately replace them with new rails. In this case the rail is only connected to the immediate base plate.

The important functions of fastenings are: to retain the rail against the sleeper; to resist the vertical, longitudinal, lateral and overturning movements of the rail. In addition they connect sections of rail to permit safe and smooth train operation.

Elastic fasteners: They have four primary components and are designed to perform a specific function within the fastening system.

- *Clip or spring*- to apply an appropriate clamping force (toe load) to the base of the rail either by bolt or screw; or alternatively by driving clips into a cast-in shoulder.
- *Anchor*- to hold the clip or spring to the tie, and is cast-in during the tie manufacturing process. The anchors are bolts or screws, and cast-in shoulder (to driven clip).
- *Tie pad*- to properly attenuate the loads exerted by the rail onto the tie.
- *Insulator- to* properly insulate the fastening system from electrical current and from vibration/noise to facilitate reliable operation of the signal system.

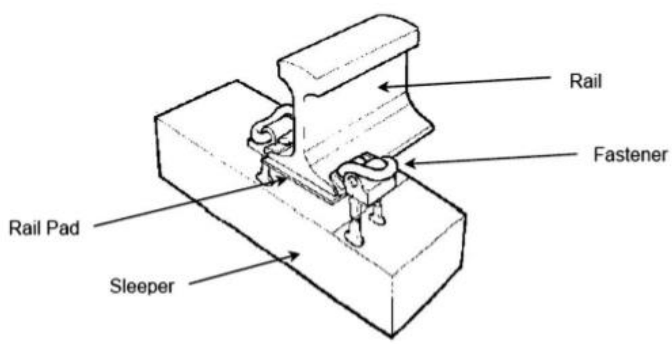


Figure 3-8: Typical fastening system for concrete sleepers

Stiffness of Fastening Systems

Stiffness closely relates to the degree of wear fastening system components experience. It is expressed as the unit of applied force per unit of deflection (lb/in.), and directly impacts the fastening system's long-term performance under repeated axle loading.

Selection of Fastener Types

The choice of fastening is greatly depend on the properties and structure of the sleeper, in terms of: cost and adjustability, fastening force and bearing ability. Spring clips are an integral part of the concrete tie system for ballasted track structures. But, when spring clip fastenings are in use, additional inspections are necessary to provide sufficient warning of loss of track gauge and/or rail retention shall be instituted.

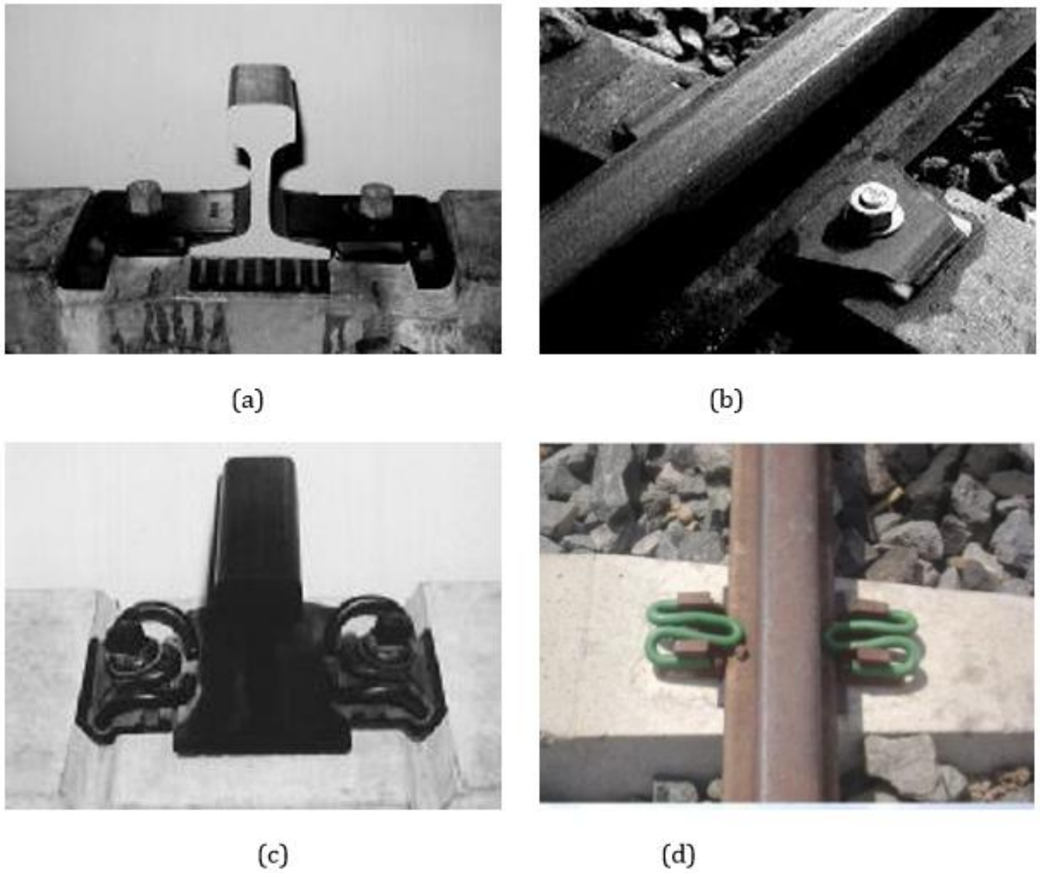


Figure 3-9: Different elastic rail fastening (a) Japan leaf spring, (b) France leaf spring, (c) Germany Vossloh wire spring, and (d) Ethio-Djibouti railway line Pandrol fast clip

Rail pads are required between the rail seat and concrete sleeper surface to provide: sufficient resiliency for the rail/sleeper system; damping of wheel induced vibrations; prevention or reduction of rail/sleeper contact attrition; and Electrical insulation for the track signal circuits. Resilient pads are not used with wood sleepers because the wood itself provides resiliency.

3.4. Track Ballast

It is the upper stratum of the substructure which supports the rails and sleeper. The most important functions of ballast are: to distribute the load from the sleepers, to damp dynamic loads, and to provide lateral resistance and rapid drainage.

3.4.1. Components of Track Ballast

The ballast can be classified in four zones

I. crib, or the ballast in between the sleepers;
II. shoulder, the material beyond the sleeper ends down to the bottom of the ballast layer;
III. top ballast, the upper portion of load bearing ballast layer which is disturbed by tamping;
IV. Bottom ballast, the lower portion of supporting ballast layer which is not disturbed by tamping, and which is generally the most fouled portion.

Only the top and bottom ballast distributes the load transmitted from a sleeper down to the sub-ballast and further on to the subgrade. The role of crib ballast and shoulder ballast is mainly to provide minimum confinement against lateral movement.

3.4.2. Ballast Material

The best choice of material will not necessarily be the highest quality material or the material that has the lowest delivered cost but includes cost effective choice considering: traffic, environmental conditions, and cost of material delivered to the site

Traditionally, good ballast materials are angular, crushed, hard stones and rocks, uniformly graded, free of dust and dirt, not prone to cementing action, and that have high specific gravity.

- Angular stones are preferable to achieve the best interlock properties
- Crushed aggregates have good shear resistance and internal friction
- Hard stones and rocks have good durability.
- Fairly uniformly graded ballast is best to achieve adequate drainage whereas uniform support may be achieved by well-graded ballast.
- Ballast should be free from dust and dirt to avoid health risk of the operators. - Cementing property of aggregate may decrease permeability.
- The bulk density and specific gravity control the stability of the track (especially lateral stability of curved track), and should be maximized without significant reduction in drainage.

To ensure that ballast is of good quality, ballast needs to be tested after the manufacturing process at the quarry; like gradation, grains size, shape, hardness, abrasion resistance and mineral composition. Due to the lack of universal agreement on the specifications for ballast materials, availability and economic considerations have been the main factors considered in the selection of ballast materials

3.4.3. Problems Track Ballast

Serious problems of track ballast mainly are related to the lack of adequate drainage of the ballast layer;

A. ***Ballast Degradation:*** Ballast particles can suffer degradation due to the action of traffic and maintenance operations in broadly two ways: Either edge can become rounded and lose their interlocking effect or Particles can break or crush under repeated loading.
B. ***Ballast Fouling:*** The fouling of the ballast is regarded as the proportion of fines expressed as a weight percentage of the total sample exceeding the permissible proportion according to the technical specifications due to damage and contamination of ballast after long term service
C. ***Pumping:*** Pumping track sections are those that exhibit pronounced movements under the dynamic loads of passing trains and produce visible amounts of muddy water or slurry from the ballast structure or subgrade.
D. ***Settlement:*** may not be a problem if it occurs uniformly along the length of the track; and influenced by; large trainloads, the number of load cycles, and high speed of trains.

3.5. Track Subballast

The subballast is the second structural layer of granular material that has the main functions to distribute stress from ballast to subgrade. it helps the ballast to reduce the stress on subgrade, maintains separation between the ballast and subgrade particles, and plays an important role in track drainage. It keeps the subgrade from penetrating up into the ballast while wet and under pressure

Alternatives or supplements to sand/gravel subballast materials:

- ***Filter fabric (geo-textile):-*** but investigations have shown that fabric is not generally desirable
- ***Asphalt concrete:-*** but the economics will probably limit asphalt to special cases

3.5.1. Sub-Ballast Material

Requirements for crushed rock aggregate of subballast materials are: maximum grain size suited for equalising the stress distribution; fairly broadly graded grain size distribution enabling good compatibility and to fulfill filter requirements; lowest possible fines content; high impact strength; high abrasion strength; lowest possible susceptibility to chemical and frost weathering; cubic, sharp-edged grain shape; high grain surface roughness; and high water permeability and low water retention of fines separating from material.

The durability requirements of subballast are not as severe as for ballast because the subballast particles are smaller and the stresses are lower.

3.6. Design of Railway Track Structure

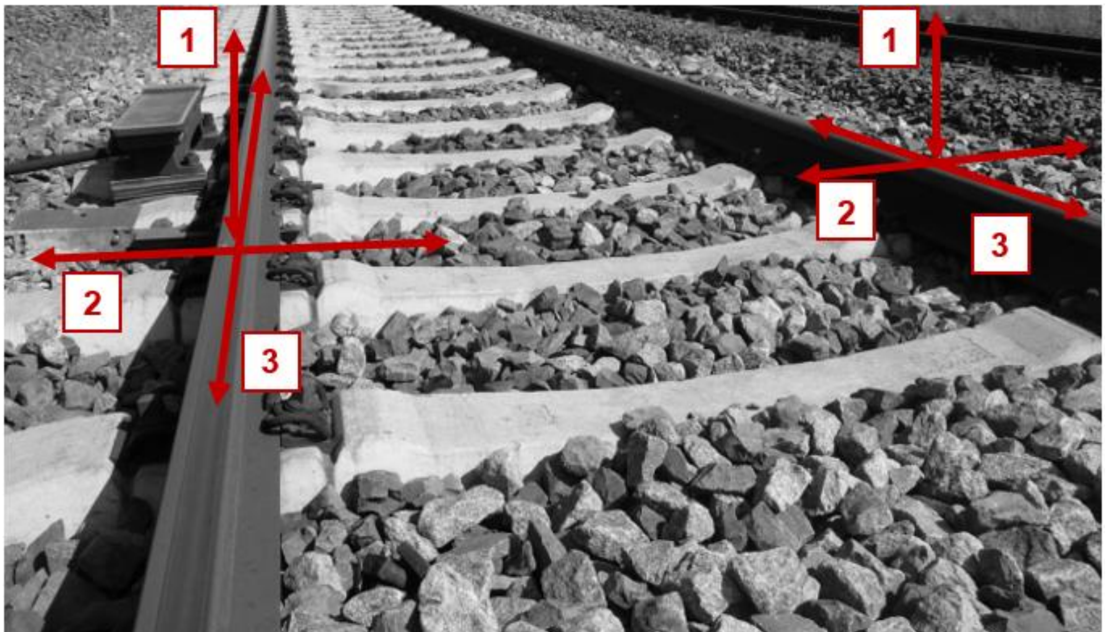
For a long time design of railway tracks has been a matter of learning from experiences. New insights and new techniques are now coming to a balanced and even optimum track design. The number of types of track structures which are used nowadays is mainly reduced for reasons of constructability, maintainability, and reliability. Moreover tracks are generally designed and built with a large reserve in order to avoid a possible failure during operation, or to meet new operational needs in the future.

3.6.1. Track Components and Loading

The loads from the trains are applied by the vehicles onto the rails in the track. The vertical, lateral and longitudinal forces resulting from the carrying, guiding and moving between wheel and rail, as well as from temperature, are shown in the adjacent figure. They must be taken and dissipated from the carrying system track-permanent way-subsoil (extended infrastructure) without damage to the environment, in order to guarantee safe operations with a high level of comfort at all times.

The permanent way, made up of track grid and ballast bed or support plate, ensures by a suitable frame stiffness as well as its resistance to slipping, longitudinal and transverse displacement, that it can accept the transverse and longitudinal forces.

The forces imposed on the track structure could be classified as mechanical (both static and dynamic) and thermal. The sources of these forces are from:



1 **Vertical forces**
- → Wheelset forces
- → Additional forces due to running in curves, discontinuities in the track, rough riding

2 **Horizontal forces (transverse)**
- → Guiding force from sinusoidal running
- → Centrifugal forces in curves
- → Wind force
- → Additional forces from discontinuities in the track

3 **Horizontal forces (longitudinal)**
- → Friction force for driving/braking
- → Friction force when rolling
- → Longitudinal forces from temperature changes in continuously welded track

Figure 3-10: Interactions between vehicle and track systems

3.6.1.4. Basic assumption of Winkler support model

- The two continuous parallel rail as abeam with infinite length on elastic foundation
- the rail which are fixed at regular interval on to sleepers supported from below and from
- the side by a medium which cannot be deformed
- the ballast bed rests on a formation which also cannot be deformed
- vehicle wheel load equally distributed to the two continuous parallel rail
- the track weight should not be considered (ignored)

In elementary calculations it is usually presupposed that the Winkler hypothesis applies to track support, this hypothesis was formulated in 1867 and reads: at each point of support the compressive stress is proportional to the local compression. This relation can be written as (Esveld, 2001):

$$\sigma = C * w \tag{3-1}$$

Where:

- *σ = local compressive stress on the support [N/rn2];*
- *w = local subsidence of the support [m];*
- *C = foundation modulus [N/rn3].*

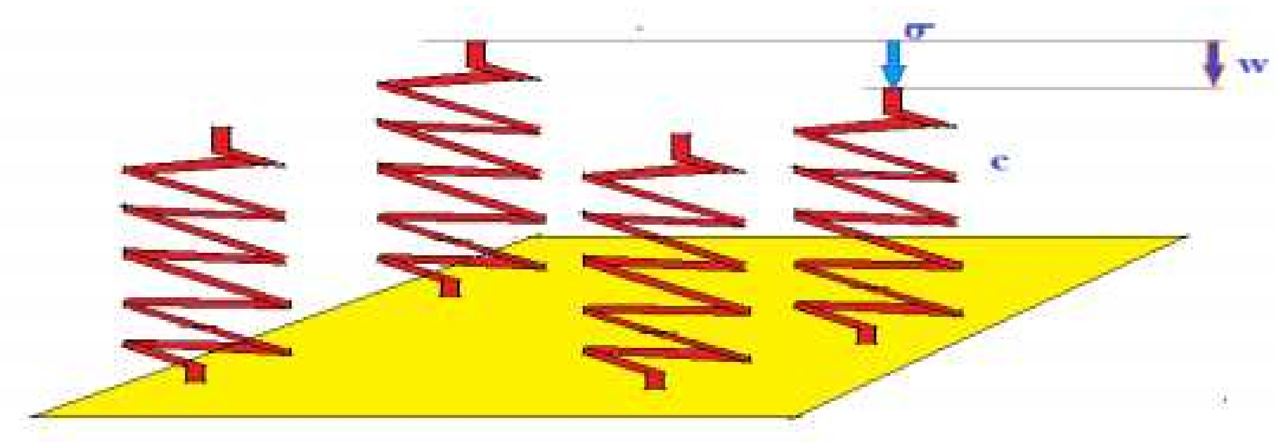


Figure 3-11: Winkler support model

3.6.1.5. Beam on elastic foundation model

Under the vertical wheel force or wheelset force $P = P_{stat} + P_{dyn}$ the rail or track experiences an elastic deformation s (spring deflection). This acts due to the flexural strength of the rails as an elastically mounted bearer in a way that the wheel force can be transferred to several sleepers (load distribution) and a reduction in the supporting point force and the ballast pressure.

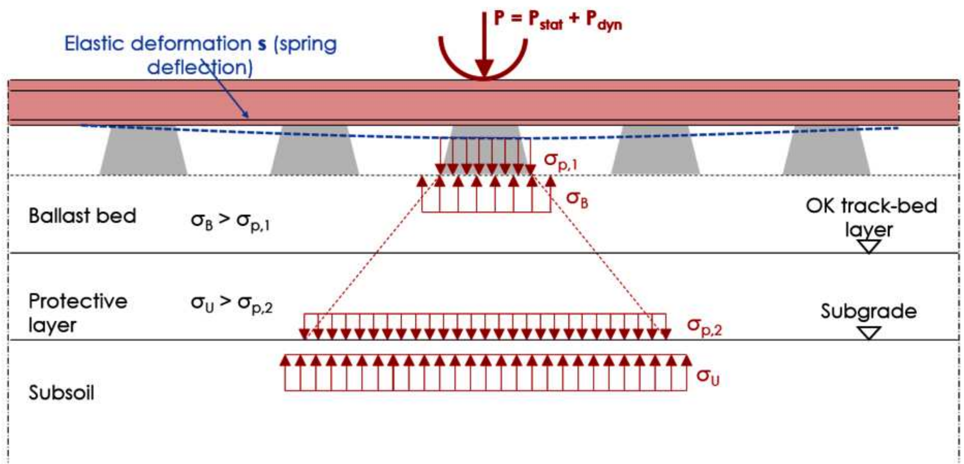


Figure 3-12: Bearing system superstructure and subgrade (Kerr, 2010)

Let us consider an infinitely long track (CWR) with bending stiffness EI which is continuously supported by an elastic foundation with foundation coefficient K loaded a wheel load Q at x = 0. This beam calculation was first proposed by Zimmermann. To derive the formula for the deflection y(x) of the beam, we first write down the equilibrium conditions of the beam element.

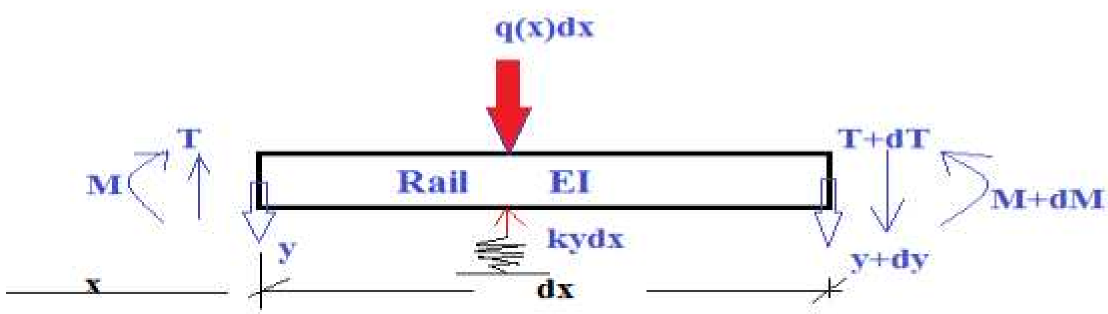


Figure 3-13: Equilibrium condition of beam element

Equilibrium requires:

$$\sum Fv = 0$$

T + kydx - (T + dT) – q(x)dx = 0

kydx - dT – q(x)dx = 0

q(x)dx = kydx - dT

q(x) = ky- dT/ dx

Where:

From elastic liner equation $d^2y/dx^2 = -M/EI$

$$M = -EI\, d^2y/dx^2$$

$dT/dx = d^2M/dx^2$

$dT/dx = -EId^4y/dx^4$

$q(x) = ky - (-EId^4y/dx^4)$

From these equations the differential equation for the rail deflections become:

3-2

$$q(x) = \frac{EId^4y}{dx^4} + ky$$

Where:

- *E = Young's modulus of rail steel*
- *I = moment or inertia of one rail with respect to the horizontal centroidal axis*
- *y = vertical track deflection*
- *q = vertical load distribution (wheel loads) on the rail*
- *x = point on rail axis*
- *k = elastic modulus of rail support (truck modulus), for one rail*

Since we deal only with point loads, the distributed load q(x) will not be considered here (q=0).therefore the general solution of the differential equation is

3-3

$$y = C_1 e^{kx} \cos kx + C_2 e^{kx} \sin kx + C_3 e^{-kx} \cos kx + C_4 e^{-kx} \sin kx$$

After substituting the boundary condition in the general equation, than the magnitude of the deflection (y), at point x of track, rail support force (R) and bending moment (M) can be expressed by the following expressions:

3-4

$$y(x) = \frac{Pk}{2u} e^{-kx} (\cos kx + \sin kx) = \frac{Pk}{2u}\, \eta(kx)$$

3-5

$$M(x) = \frac{P}{4k} e^{-kx} (\cos kx - \sin kx) = \frac{P}{4k}\, \mu(kx)$$

3-6

$$R = q(x) * a = u * y(x) * a = \frac{Pka}{2} e^{-kx} (\cos kx + \sin kx) = \frac{Pka}{2}\, \eta(kx)$$

Where:

- *Stiffness ratio coefficient (K): a coefficient between rail foundation stiffness (μ) and the rail bending stiffness (EI)* $k = \sqrt[4]{\frac{\mu}{4EI}}$
- *Characteristic length* $L = \sqrt[4]{\frac{4EI}{\mu}} = \frac{1}{k}$
- *a -sleeper spacing*
- *u -rail foundation stiffness=D/A*
- *D–rail support stiffness*
- $P^d = (1+\theta)\,P =$ dynamic wheel load
- $y_{max} = \frac{P}{2Lu}$
- $M_{max} = \frac{P}{4k}$

Dynamic amplification factor (θ) and maximum design load

It is common practice to carry out a strength or fatigue calculation for a static load system or often a single wheel load, using the longitudinal beam theory, according to which the dynamic effects are taken into account by a speed coefficient or dynamic amplification factor. The effect of running speed on load is in reality highly complex because of the dynamic interaction between vehicle and track. In view of the nature of the load, it is also more correct to carry out a fatigue calculation. The Eisenmann scheme to determine the DAF (Dynamic Amplification Factor) is dependent on the train speed, the track quality, and chosen factor (t) and reads as follows

DAF- formula:

$$DAF = 1 + t\varphi ....if \;\; speed(V) \; less \; than \; 60 km/hr \tag{3-7}$$

$$DAF = 1 + t\varphi(1 + \frac{V-60}{140}) ......if \;\; 60 \le V \le 200 km/hr \tag{3-8}$$

Table 3-7: Eisenmann DAF (Dynamic Amplification Factor) approximation

Application	T	Track condition	φ
Contact stress, subgrade	1	Very good	0.1
Lateral load, ballast bed	2	Good	0.2
Rail stresses, fastenings, supports	3	bad	0.3

Where:

- *t = multiplication factor of standard deviation which depends on the confidence interval. Since the rail is so important for safety and reliability of rail traffic a value of 3 is recommended*
- *φ = factor depending on track quality,*
- *V = train speed [km/h].*
- *P^d = DAF*P*

3.7. Design-Procedures (Simplified)

3.7.1. Rail

Example 3-1: *Given conditions*

- maximum design speed 70Km/h and
- use axel load of 11t for preliminary selection of rail section
- track condition is very good
- Sleeper spacing is 0.6m

Step-1: Preliminary selection of rail section

According To International Union of Railway (UIC)

The choice of rail section mainly depends on the traffic load as well as on the intervals between renewal sessions. For a standard-gauge track, it is customary to use UIC 50 rail for low traffic load and UIC 60 for medium and heavy load.

Table 3-8: Choice of the rail section in relation to traffic load (UIC) (Vitez I., 2004)

Daily line traffic load (tons)	***≤25,000***	***b/n 25,000 and 35,000***	***≥35,000***
weight of the rail per meter length	50kg/m	- 50kg/m (for timber tie) - 60kg/m(for concrete tie)	60kg/m

According To USA Department of Defense (USADOD)

The rail road design and rehabilitation by department of defense, USA, gives recommendation on rail section selection primarily by the following formula.

3-9

$$w_o = 315 - \left(\frac{21200}{\frac{p}{1120} * a + 67} \right)$$

Where

- w_o =*Weight of rail (lb/yd)*
- p =*design wheel load (lb)*
- a =*Impact factor, where:*
- a =*1, where design operating speed is 25 mph or less than 25mph*
- a =*1.4, where design operating speed is more than 25 mph*

By using different types of wheel load for different types of train, selecting the one which is produces maximum dynamic design load and you can insert this maximum load in to the above formula, than you will get easily the preliminary section of the rail, but for this example, the maximum design speed 70Km/h and the maximum design axel load is 11t which is very high compared to the other load due to this, no need of comparisons for selection of the design load , we can use this design load as it is, so for this design load, the wheel load become 55kN (Axle load/2)

Notice:

- 1miles=1.609344km therefore 70km/h = 43.5mph
- a =1.4 since the design speed (V)>25 mph
- lb/yd=0.496kg/m

By taking the maximum wheel load (Axle load/2); P=55KN (12,125.42 lb), the rail weight will be:

$$w_o = 315 - \left(\frac{21200}{\frac{12,125.42}{1120} * 1.4 + 67} \right) = 56.95 \text{ lb/yd} = 27.9 \text{ kg/m}$$

But we have no such rail weight per meter in Chinese rail section standard so we can take the approximate rail section which is 50kg/m and according to INTERNATIONAL UINION OF RAILWAY from (table 3-8: above Choice of the rail section in relation to traffic load) it is possible to select 50 kg/m rail section for 11t design axel load.

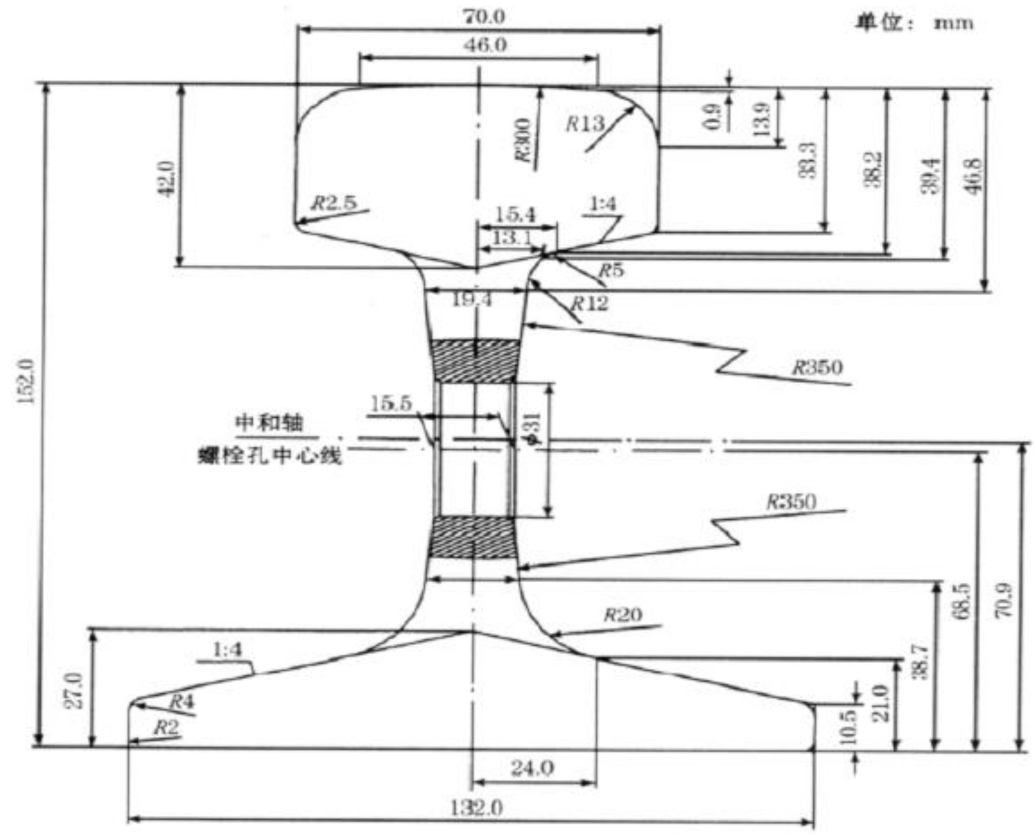


Figure 3-14: Rail profile 50kg/m

Table 3-9: Property of rail section (50kg/m)……. From GB-standard

rail style(kg/m)	50
section area (cm2)	65.8
center of gravity from bottom of rail (cm)	7.1
center of gravity from end of rail (cm)	8.1
moment inertia horizontal axis (cm4)	2037
moment inertia vertical axis (cm4)	377
cross-section coefficient of bottom (cm3)	287.2
cross-section coefficient of top (cm3)	251.3
cross-section coefficient of bottom side edge (cm3)	57.1

Step-2: Maximum design load

a. **According to Eisenmann DAF become:**

Our track condition is very good; **φ=0.1**

For application of rail stress; t=3

Design speed; V=70km/h

$$DAF = 1 + t\varphi(1 + \frac{V-60}{140}) ......if\ \ 60 \le V \le 200 km/hr$$

$$DAF = 1 + 3*0.1(1 + \frac{70-60}{140}) = 1.32$$

P^d = DAF*P =1.32*55kN=72.6kN

b. According to AREMA -2010, Maximum stresses in track occur under dynamic loading. the dynamite amplification factor for taking into consideration dynamic effect is given by:

3-10

$$\theta = \frac{33V}{100D}$$

Where:

- *V=dominant train speed (mph)*
- *D=diameter of wheel load (inch)*

Dynamic load Amplification Factor:

V=70 Km/hr =43.5mph

D=840 mm = 33.071 inch (source: Chinese manufacturer manual)

$$\theta = \frac{33V}{100D} = \frac{33*43.5}{100*33.017} = 0.434$$

The design load will be:

$$Pd = (1+\theta)P = (1+0.434)*55KN = 78.87KN$$

Therefore P^d =78.87 KN (AREMA) > P^d =72.6 kN (Eisenmann). Hence for our analysis we take the maximum design load that is P^d =78.87 KN (AREMA)

Step-3: Maximum rail bending moment

For one wheel load, the maximum rail bending moment, M_{max}, take place at the wheel and it is provided by the following equation. $M\max = \frac{P^d}{4K}$, Talbot equation, from china standard for 50kg/m rail section the modulus of elasticity and the moment of inertia from table 3.2: become E= 207Gpa and I=2037cm^4 and from the classic ballast track we can use the stiffness of ballast and fastening, then we can calculate the track modulus (u)

Table 3-10: Classic ballast track stiffness and dumping

Track component	Stiffiness	Damping
UIC-50kg/m		
Rail pads(per rail)	100E3kN/m	15kNs/m
Sleeper per rail (for concret sleeper)	Infinity	
Ballast bad (distributed per rail)	27E3kN/m (per sleeper)	12.3kns/m

Sleeper spacing is 0.6m (given)

$u = D/a$

$$\frac{1}{D} = \frac{1}{D_f} + \frac{1}{D_s} + \frac{1}{D_b}$$

Where: D_f=100kN/mm=100e6N/m; concrete sleeper, D_s=∞; D_b=27 kN/mm=27e6N/m

$$\frac{1}{D} = \frac{1}{100e6} + \frac{1}{\infty} + \frac{1}{27e6\text{N/m}} \Rightarrow D = 21.27\text{ e6N/m}$$

$$u = \frac{D}{a} = \frac{21.27e6N/m}{0.6m} = 35.45\text{ e6N/m2}$$

Stiffness ratio coefficient (K):

$$k = \sqrt[4]{\frac{\mu}{4EI}} = \sqrt[4]{\frac{35.45\,\text{e6N/m2}}{4*207e9N/m^2*2.037e-5\,m^4}} = 1.2m^{-1}$$

Thus the maximum moment will be:

$$M\max = \frac{78.87KN}{4*1.2m^{-1}} = 16.43KNm$$

Step-4: Maximum dynamic bending stress

The rail bending stress is usually calculated at the centre of the rail base, but the stress at the lower edge of the rail head may be critical if the vehicles impose high guiding forces during curving between the wheel flange and rail head. For design purposes, the maximum dynamic bending stress in the rail is (AREMA Vol.4 chap.16, 2009):

3-11

$$\sigma_{\max} = \frac{M_{\max}*c}{I} = \frac{M_{\max}}{Z_b}$$

Where:

- $\sigma_{\max}$ =The maximum stress on the rail
- *c = distance from neutral axis to rail base (mm);*
- *Z_b = section modulus for rail base, I/c (properties of rail section); and*
- M_{max} = the maximum design moment

Z_b=287.2cm^3............... (From table)

Z_b=2.872E-4m^3

$$\sigma_{\max} = \frac{16.43KNm}{2.872\text{E - 4m3}} = 57.207\text{MPa}$$

Step-5: Verification of rail section

To verify the rail section (50kg/m), according to bending stress, the listed allowable stresses are mandatory. Therefore by using this stress we can verify our rail section:

Allowable bending stress typically:-

- 32,000 psi for jointed rail
- 25,000 psi for continuously welded rail (CWR)

To convert this allowable bending stress to metric unit N/m2 multiply by 7,030.696N/m2 (conversion factor) Than the allowable bending stress become;

- 221,982,272N/m^2=222MPa for jointed rail
- 175,767,400N/m^2=176MPa for continuously welded rail (CWR)

Therefore the verification of the rail section according to bending stress

a < 222MPa……………………………… ok!

This means the calculated maximum design stress is less than the allowable stress (222Mpa), hence the preliminary rail section is enough!

Step-6: Ultimate tensile strength of rail

The concentrated load between wheel and rail causes a shear stress distribution in the rail head at some depth a maximum which may give rise to fatigue fracture in the rail head (*Esveld , 2001*). The contact problem is most serious in the case of high wheel loads or relatively small wheel diameters. According to the Hertz theory, the contact area between two curved elastic bodies such as wheel and rail head is generally ellipsoidal and the contact stress distribution is semi ellipsoidal.

(a) **(b)**

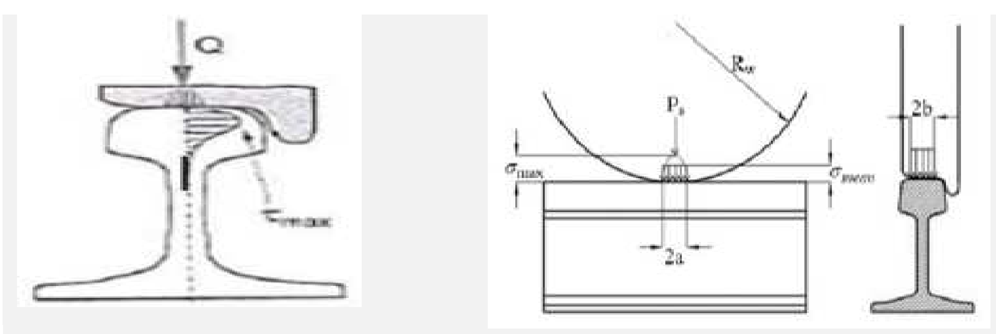


Figure 3-15: (a) Shear stress distribution in the rail head (b) Assumed contact pressure distribution between wheel and rail According to Eisenmenn

Step-7: Ultimate tensile strength of rail steel against allowable stress

The average or mean contact stress on the rail head is given by,

$$\sigma_{mean} = 1374\sqrt{\frac{Q}{r}} \tag{3-12}$$

Where:

- σ_{mean} *= the mean contact pressure (N/mm2)*
- *Q= Effective wheel load (KN) and*
- r = wheel radius (mm).

Q=55 KN; r =840mm/2=420mm

$$\sigma_{mean} = 1374\sqrt{\frac{Q}{r}} = 1374\sqrt{\frac{55KN}{420mm}} = 497.21\,\mathrm{MPa}$$

According Eisenmann, he proposed that, the limit value for allowable contact stress as a percentage of the ultimate tensile strength of rail steel, by considering required fatigue strength for rail steel. Based on this assumption, he suggests:

$$\sigma_{all} = 0.5\sigma_{ult} \tag{3-13}$$

Therefore, at limiting condition,

$$\sigma_{mean} \leq \sigma_{all} = 0.5\sigma_{ult}$$

$$\sigma_{ult} \geq 1.97\sigma_{mean} = 1.97 * 497.21\,\mathrm{MPa} = 979.5\,\mathrm{MPa}$$

a. According to china standards (GB)

Thus, the numerical result shows that the tensile strength of the rail should be at least 979.5MPa to fulfil the requirement of wheel contacts stress. According to china standard: mechanical property of rail, for ultimate tensile strength of 979.5 MPA it become one of the two steel grade U75V and U76NbRE for our case we can use U75V steel grade.

Table 3-11: Mechanical properties of rail

Steel grade	Mechanical properties	
	Tensile strength R_m, N/mm^2 （min.）	Elongation A % （min.）
U74	780	10
U71Mn	880	9
U70MnSi		
U71MnSiCu		
U75V	980	9
U76NbRE		
U70Mn	880	
When the test pieces is taken from hot saw sample rails, the results of A can smaller 1% (absolute value) than the given value.		

Table 3-12: Steel grade and chemical composition GB

Steel grades	Chemical composition %							
	C	Si	Mn	S	P	V[a]	Nb[a]	RE
U74	0.68~0.79	0.13~0.28	0.70~1.00	≤0.030	≤0.030	≤0.030	≤0.010	—
U71Mn	0.65~0.76	0.15~0.35	1.10~1.40	≤0.030	≤0.030			—
U70MnSi	0.66~0.74	0.85~1.15	0.85~1.15	≤0.030	≤0.030			—
U71MnSiCu	0.64~0.76	0.70~1.10	0.80~1.20	≤0.030	≤0.030			—
U75V	0.71~0.80	0.50~0.80	0.70~1.05	≤0.030	≤0.030	0.04~0.12		—
U76NbRE	0.72~0.80	0.60~0.90	1.00~1.30	≤0.030	≤0.030	≤0.030	0.02~0.05	0.02~0.05
U70Mn	0.61~0.79	0.10~0.50	0.85~1.25	≤0.030	≤0.030		≤0.010	—
[a] Except V in U75V and Nb in U76NbRe are additional elements , Nb and V are residual elements in other steel grades.								

b. According to international union of railway (UIC)

For the ultimate tensile strength which is greater than or equal 979.5MPa, from table 3.8 you can select the greater steel grade, to become more safer but, as we can see from the table, 979.5MPa is greater than the first row of tensile strength that is between 680-830 so that we are forced to use steel grade R900A with C (0.6-0.8), Mn (0.8-1.3) and Si (0.1-0.5) in row two.

Table 3-13: Chemical composition and tensile properties of a rail steel according to UIC

Grade of steel *	Chemical composition, elements in % of mass				Tensile strength, f_{yk} (N/mm^2)	Yield Strength, f_{yd} (N/mm^2)
	C	Mn	Si	Cr		
R700	0.4 - 0.6	0.8 - 1.25	0.05 - 0.35	-	680 - 830	380 - 460
R900A	0.6 - 0.8	0.8 - 1.3	0.1 - 0.5	-	880 - 1030	480 - 510
R900B	0.55 - 0.75	1.3 - 1.7	0.1 - 0.5	-	880 - 1030	480 - 510
R1100**	0.6 - 0.82	0.8 - 1.3	0.3 - 0.9	0.8 - 1.3	≥ 1080	≥ 650

Step-8: Ultimate tensile strength against maximum shear

The maximum shear stress on the rail head is given by:

3-14

$$\tau_{\max} = 412\sqrt{\frac{Q}{r}} \cong 0.3\sigma_{mean};$$

Where: - Q (in KN), r (in mm) and $\tau_{\max}$ *in N/mm*^{2}.

$$\text{Therefore } \tau_{\max} = 412\sqrt{\frac{55KN}{420mm}} = 149\ \text{N/mm}^2 = 149\ \text{MPa}$$

The allowable shear stress on the rail, taking account of the fatigue nature of the load, is given by:

$\tau_{all} \cong 0.3\sigma_t = 0.3 * 979.5\text{MPa} = 293.85\text{Mpa}$

Hence $\tau_{\max}$ = 149 MPa < τ_{all} = 293.85Mpa Therefore, the selected rail property satisfies the requirement to resist the maximum shear stress on the rail head.

Step-9: Check for serviceability

AREMA has proposed a limiting range for the magnitudes of vertical rail deflections. According to this recommendation, extreme vertical rail deflections should be kept within the range of 3.175 to 6.35 millimeters.

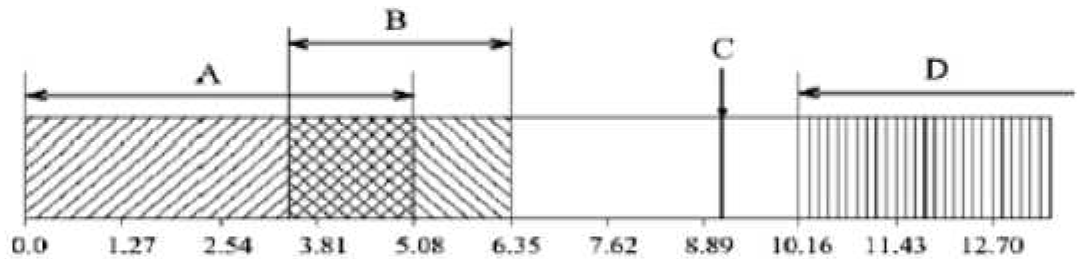


Figure 3-16: Vertical track deflection range

Domains indicated in figure above: are described as follows:

- A: Deflection range for track which will last indefinitely.
- B: Normal maximum desirable deflection for heavy track to give requisite combination of Flexibility and stiffness.
- C: Limit of desirable deflection for track of light construction (with rails weigh < 50 kg/m)
- D: Weak or poorly maintained track which will deteriorate quickly.

It should also be noted that values of deflection in fig.3-16 do not include any looseness or play between rail and pad or pad and sleeper. In addition, these values represent deflections directly under the wheel load. According to AREMA, the present design rail section track can be categorized under domain-A, which has between the ranges of 0.00mm up to 3.00 mm.

$$y_{\max} = \frac{P^d K}{2u} = \frac{78.87KN * 1.2m^{-1}}{235490KN/m^2} = 1.34mm$$

Therefore the calculated maximum deflection y_{max} = 1.34 mm< 6.35 mm (AREMA max. Deflection)..ok! Hence, the given track stiffness and rail section is adequate for the design maximum load.

3.7.2. Sleepers

The choices of concrete sleepers, more widely used in the world, are because of different reasons; basically it is according to economic advantages. In addition, in Ethiopia steel is relatively highly costly; and the concrete sleepers are not affected very much by either climate or weather, made concrete sleeper preferable than others.

Average pressure at bottom of sleeper

The exact contact pressure distribution between the sleeper and the ballast and its variation with time is an important item in the structural design of sleepers. The contact pressure between tie and ballast for a well-maintained track is largest at the rail seat and smallest at the tie center (AREMA, 2009).

The values of pressure distribution are a function of the accumulated traffic, flexibility of the tie, tie base dimensions, center-to-center tie spacing, compactness of the ballast, and stiffness of the subballast, ballast, and subgrade. In order to calculate contact pressure between the sleeper and ballast several approaches have been developed.

The allowable sleeper-ballast contact pressure shall be underestimated (i.e. AREMA approach) to avoid over stress on ballast and the effective length of ballast support beneath each rail seat (bearing area) of Australia standard approach shall be taken The ballast pressure P_a is based on a uniform pressure distribution beneath each rail seat and shall not exceed 750 kPa for high-quality, abrasion-resistant ballast (AS1085.14, 2003).

Table 3-14: Maximum ballast-sleeper pressure for standard gauge (AS1085.14, 2003)

Distance between rail centers (g)	Length of ballast support beneath each rail seat (L) (m)	Maximum ballast bearing pressure (P_a) (kPa)
g > 1.5 m	L=l-g	$P_a = \frac{q_r}{b \times (l - g)}$

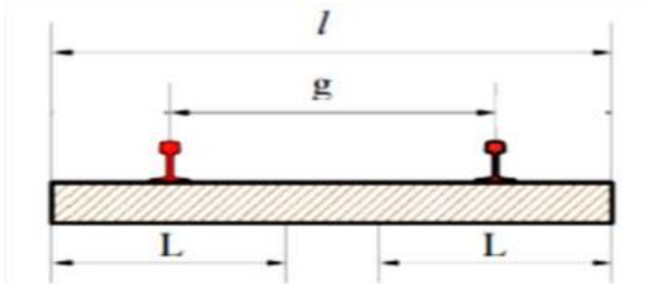


Figure 3-17: Effective length (area) of sleeper support at rail seat (Sadeghi J. and P. Barati, 2010)

For the purpose of simplification, it would be more practical and widely accepted to consider only the effect of some of parameters for example, the value of vertical rail seat load as a function of sleepers' type and spacing (Sadeghi J. and P. Barati, 2010).

$$q_r = P_s * DF(1+\varnothing) \qquad \text{3-15}$$

Where:

- *q_r-rail seat load, b- sleeper base width, l- length of sleeper,*
- *g- center-to-center spacing of track gauge and*
- *P_{all}-allowable average intensity of pressure on ballast.*
- *Ps = Wheel load in pounds (KN);*
- *Ø = Impact factor in percent (Ø =2 is assumed);*
- *DF = Distribution factor*

Table 3-15: typical values (AREMA, 2010)

Sleeper type	*Allowable stress on ballast surface(P_{all}), KPa*
Wooden	450
Concrete	586

Dimension and Spacing of Sleeper

The range of length, base width, and height of sleeper and also spacing between sleepers can be adopted from AREMA directly and the actual value can be determined by optimization analysis and /or design requirement of resisting impact of applied load.

Provision of sleeper spacing shall be generally based on optimizing the quality and dimension of sleeper to carry load and the number of sleeper to transfer the load efficiently to ballast without excessive damage. The optimization can be done on the investment cost of quality and size of sleeper and number of sleeper per kilometer track.

Optimization of Sleeper Dimension

The sleeper spacing, sleeper base width and sleeper length can be optimized in consideration of satisfying the allowable bearing pressure. For this optimization analysis, it needs a full design of sleeper to estimate cost.

Steps to optimize parameters or dimensions (sleeper spacing, sleeper average base width and sleeper length) of concrete sleepers

i. Selecting combination of sleeper parameters that satisfy allowable bearing pressure for specific design wheel load
ii. Computing the single concrete sleeper cost including rail pad and fasteners by assuming profile and height of sleeper
iii. Computing the number of concrete sleepers per kilometer track length for different sleeper spacing
iv. Computing the total cost of concrete sleepers per kilometer track length
v. The combination of sleeper parameters with minimum cost is selected

Example 3-2: *Determine spacing, width, length, and depth of PSC sleeper by taking the following assumptions as base case.*

- *18 tonne axle load (90 KN wheel load)*
- *200 mm height for sleepers' combination with 520 mm sleeper spacing to avoid bending effect and an increment of 5 mm for each increment of 50 mm length*
- *Average top width of sleeper is proportional to base width*
- *2% cost increment for each 30 mm spacing increment to add quality on concrete sleeper to avoid any failure due to load (increasing quality of concrete mix, increasing quality of rail pad or other treatment).*
- *A cost of 6,000 birr per 1m^3 of concrete sleeper that spaced at 520 mm including fasteners and rail pads.*

Solution:

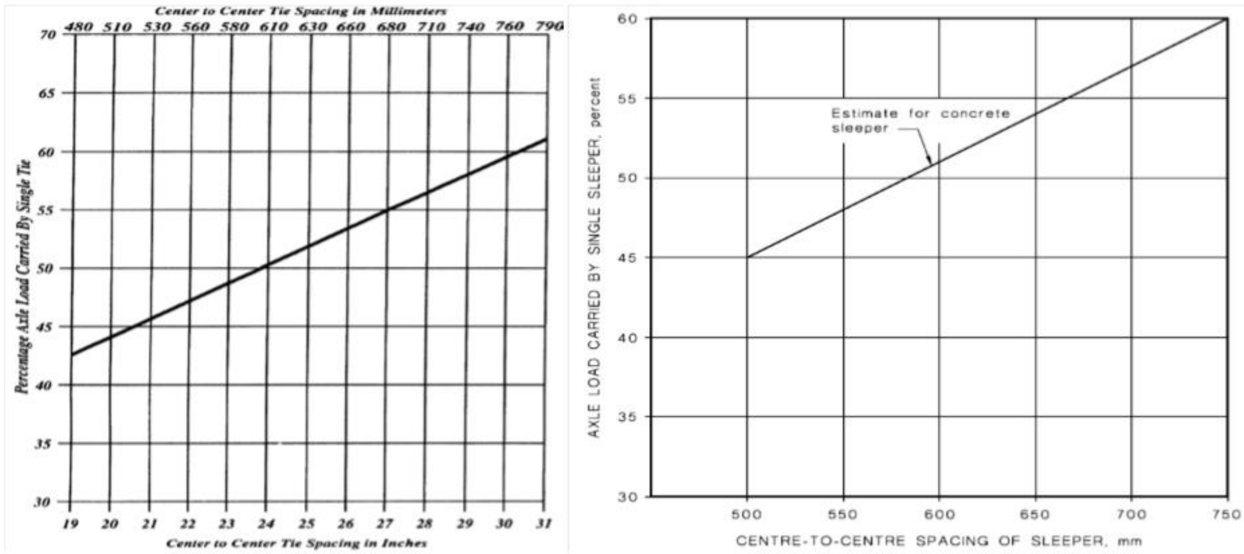
Step-1: minimum average base width;

$$b = \frac{q_r}{P_a * (l - g)}$$

$$q_r = P_s * DF(1 + \text{Ø}) = 90 * \text{DF}(1 + 2) = 270DF$$

P_a=450kpa (allowable pressure) and g = 1.5 m; Substitution of ***q_r, P_a****, and* **g** *values gives;*

$$b = \frac{270DF}{450 * (l - 1.5)}$$



(a) (b)

Figure 3-18: Estimated distribution factor of loads (a) AREMA and (b) Australia standards

Thus, minimum average base width becomes function of ***DF*** *and* ***l***

Minimum average bottom width of sleeper (mm) at a wheel load of 90 KN and allowable pressure of 450 KPa for different sleeper spacing and length is as tabulated below

Sleeper Spacing (mm)	DF	Minimum Average bottom width of sleeper, b (mm)						
		l = 2.4	l = 2.45	l = 2.5	l = 2.55	l = 2.6	l = 2.65	l = 2.7
520	0.46	307	291	276	263	251	240	230
550	0.48	320	303	288	274	262	250	240
580	0.5	-	316	300	286	273	261	250
610	0.52	-	328	312	297	284	271	260
640	0.535	-	-	321	306	292	279	268
670	0.55	-	-	-	314	300	287	275
700	0.57	-	-	-	326	311	297	285
730	-	-	-	-	-	316	303	290
Note: (-)-The value beyond the recommended range.								

By taking minimum average widths within the recommended range (b/n 200mm and 330mm by AREMA); the next step is optimization in terms of sleeper cost.

Step-2: Computing the number of concrete sleepers per kilometer track length for different sleeper spacing

The number of sleeper per kilometer track length, N = (1000/Sleeper spacing (mm)).

Step-3: The volume of concrete sleeper (V)

The volume of concrete sleeper can be calculated as:

$$V = l * h * \frac{T + b}{2}$$

Where: l- length of sleeper, b- average base width of sleeper, T- average top width of sleeper, h-height (depth) of sleeper.

Step-3: Computing the total cost of concrete sleepers per kilometer track length

The total cost of sleepers per kilometer can be calculated as

$$Total\ \cos t = V * N * 6000 birr / km$$

The quality of sleeper spaced broadly can be adjusted by multiplying the factor, f.

Where: f-quality increment factor (indicated in the base case).

Step-3: select sleeper cross section with minimum total cost per kilometer (use excel spread sheet).

	A	B	C	D	E	F	G	H	I	J
1	S (mm)	L (mm)	b (mm)	T (mm)	H (mm)	V (m3)	N (no/Km)	f	Total cost	
2	.									
3	.									

Sheet1

3.7.3. Plate Size Selection

Step-1: Selection of fastening type

The type of fastening system that shall be selected for a given railroad are mostly based on minimum cost with a good ability of fastening force, capacity of bearing ability and easily adjustable.

Step-2: theoretical design

- Proportion the size making width fit to tie and length to keep stress on tie less than or equal to the allowable
- Distance between shoulders (double shouldered plates) spaced to match rail base width
- The stress on tie must be kept less than or equal to the allowable value

AREMA recommends 200 psi, for wood tie, as allowable stress to check or calculate length of plate.

$$stress\,on\ tie = \frac{q_r}{lb} \tag{3-16}$$

Where: q_r = Rail seat load, lbs, (N); l=length of plate, in, (mm); b=width of plate, in, (mm)

3.7.4. Ballast Section Design

The design of the ballast section includes the determination of the depth of the ballast cushion below the sleeper and its profile.

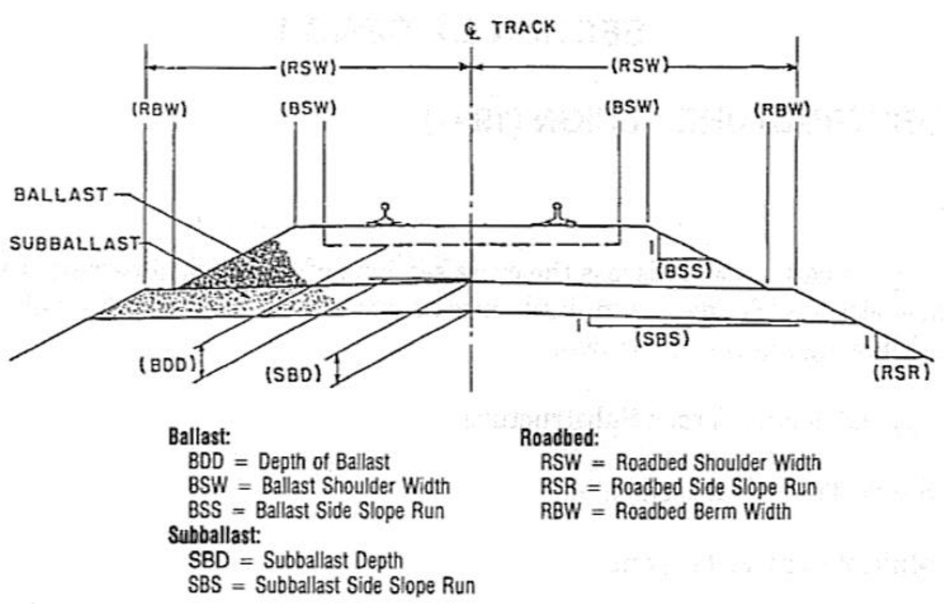


Figure 3-19: track section

Step-1: ballast depth determination (ballast + subballast)

The following equations have been developed to determine the depth of ballast (h) required from the desired pressure. **Talbot Equation** *(Selig, 1995)***:**

3-17

$$P_c = \frac{16.8 P_m}{{h_b}^{1.25}} \; where \; P_m = \frac{2q_r}{A_b}$$

Where:

- P_c=*Maximum intensity of pressure on subgrade (25 psi)*
- P_m= *Intensity of pressure on ballast = (psi)*
- q_r- *is rail seat load*
- A_b- *is tie-ballast contact area*
- h_b= *Depth of ballast below tie (inch)*

Step-2: ballast section shoulder width (BSW)

In addition to depth required, ballast section shoulder width (BSW) is proportioned to provide additional lateral strength to the track. The measure is made from the end of the cross tie to the point of beginning of the ballast side slope (S), and is made in the plane of the top of the cross tie. The ballast shoulder widths at the ends of ties in each direction are as follows (*Ernest, 1995*):

3-18

$$BSW = \frac{P * T_s}{2 * P_v} \; where \; P = 0.441 D * \Delta T$$

Where:

- P_v - *vertical load carrying capacity of shoulder (lb)*
- T_s - *Tie spacing (in)*
- *P - lateral force (lb) produced by continuously welded rail on curved track as a result of changes in temperature*
- *D = degree of curve (°) and ΔT = Temperature change*

Note: The side slope (S) run component of the ballast section is also proportioned to provide confining pressure to that part of the ballast section expected to transmit the vertical load from the bottom of the cross tie to the top of the sub-ballast and a value of 2:1 is commonly used (AREMA)

Step-3: Sub-ballast Depth (h_{sb}) Design

During structural analysis, the sub ballast layer is considered as part of the total ballast depth; and may comprise up to 40% of the total ballast thickness on main running tracks and up to 50% on auxiliary tracks. Minimum subballast section depth is 150 mm from experience of AREMA.

Generally, required depth shall be depending upon the: maximum speed of trains, maximum axle loads carried and gross annual tonnage expected; but it shall be greater than the recommended limit subballast depth.

EXERCISE 3-1:

1. Due to rapidly growing urbanization, in Ethiopia, space in urban areas is limited; and road or pedestrian traffic may also need to use the area where the railway track is installed. What is your insight if the current ballasted track, in Addis LRT, was done as covered track in prior?
2. If you are a committee member intended for standardization of guidelines for railway system in Ethiopia, which track type will you prefer based upon the factors such as; topography, climatic variation, and availability of construction material?
3. As a resident engineer; which things and how you will checkout to approve well completion of ballast layer
4. By sketching out layers of track structure, discuss how pressure distribution due to live load as well as dead load coming from trains and the super structure finally is taken by the subgrade.
5. Suppose it is required to connect point A in railway line with point B; preliminary engineering study is undertaken and demands detail design of track structure. The HXD1C electric locomotive model, which can draft 5000-5500 tons at a gradient of not more than 12%, is suggested. The train has maximum load bearing capacity of 23tons per axle with 138tons loco-weight. It has four single and two tandem axles along with wheel diameter of

1.25m. The track is required to have a desired deflection for heavy truck to give request combination of flexibility and stiffness. This is from the estimation that this track would have accumulated tonnage of ten million metric tons of freight and five million passengers annually in both sides with maximum speed of 120km/hr in a standard gauge. (Assume any necessary data)

a. Select rail weight and section
b. Determine spacing, width, length, and depth of PSC sleeper (by optimization technique)
c. Determine depth of ballast (ballast + subballast)
d. Check for track deflection using KENTRACK software

MODULE-FOUR

4. RAILWAY TUNNELLING

Introduction

A tunnel is a closed or roofed structure carrying a road through, or under an obstacle. This obstacle may be anything in the path of a preferred road alignment such as a mountain, a body of water, a building or a complete development. A short tunnel is also termed an underpass, but in general any covered length of road over 90 meters long should be treated as a tunnel.

The history of tunnels is very old. The first tunnel was constructed about 4000 years ago in Babylon to connect two buildings. The first railway tunnel in the world was constructed at the end of the nineteenth century to connect Switzerland and Italy. The cross section of the tunnel was in the shape of a horseshoe and its length was about 20 km.

The first 'tube' underground railway tunnels built in London using 'Greathead' shields were commenced in 1886 and now form part of the City branch of the London Underground Northern Line.

Tunnel construction methods, both past and present, have been practiced and modified using different methods, such as Classical methods, primarily common in Belgian, English, German, Austrian, Italian, and American systems. These methods had much in common with the primary mining/tunneling methods used until the last half of the 19th century. Today, however, improved technology have introduced new techniques, such as mechanical drilling/cutting, cut-and-cover, drills, blasts, shields, and tunnel boring machines (TBMs).

Figure 4-1: The tunnel works at Weldiya-Mekelle railway project

Tunnel design is a multi-disciplinary activity requiring geometric design, structural design, electrical and mechanical engineering, fire safety engineering and communications engineering. Interaction of the specialist designers in each of these areas is necessary to obtain the best solution. Decisions on the location and form of tunnels must only be undertaken with the advice of these specialist designers.

1.1. Need of Tunnels

The need of constructing a tunnel may arise because of one or more of the following considerations.

i. A tunnel may be required to eliminate the need for a long and circuitous route for reaching the other side of a hill, as it would considerably reduce the length of the railway line and may also prove to be economical.
ii. It may be economical to provide a tunnel instead of a cutting, particularly in a rocky terrain. Depending upon various factors, a rough calculation would indicate that for a small stretch of land the cost of constructing a tunnel is equal to the cost of a cutting in a rocky terrain.
iii. In hills with soft rocks, a tunnel is cheaper than a cutting.
iv. In metropolitan towns and other large cities, tunnels are constructed to accommodate underground railway systems in order to provide a rapid and unobstructed means of transport.
v. A tunnel constructed under a river bed may sometimes prove to be more economical and convenient than a bridge.
vi. In the case of aerial warfare, transportation through tunnels provides better safety and security to rail users compared to a bridge or deep cutting.
vii. The maintenance cost of a tunnel is considerably lower than that of a bridge or deep cutting.

However, the construction of tunnels is also disadvantageous in certain ways, as enumerated here.

(a) The construction of a tunnel is costly as it requires special construction machinery and equipment.
(b) The construction of a tunnel involves the use of sophisticated technology and requires experienced and skilled staff
(c) It is a time-consuming process.

1.2. Tunnel Alignment and Gradient

A precise and detailed survey is necessary before setting the alignment of a tunnel on the ground. A small error in setting the alignment would result in the two ends never meeting at all. When starting work, both the ends of the tunnel as well as the center line are marked with precision on the ground so that the correct length of the tunnel can be determined. An accurate survey is then carried out to ensure that the center line of the alignment and the levels are transferred properly to their underground positions. The following points require special attention when deciding the alignment and gradient of a tunnel.

a) The alignment should be straight as far as possible since normally such a route would be the shortest and most economical.
b) The minimum possible gradient should be provided for a tunnel and its approaches.
c) Proper ventilation and adequate lighting should be provided inside the tunnel.
d) The side drains in a tunnel should be given a minimum gradient of 1 in 500 for effective drainage. In longer tunnels, the gradient should be provided from the center towards the ends for effective and efficient drainage.

Level of tunnel

A high level tunnel is much shorter and reduces geological risk (because of the reduced cover). On the other hand, the operation is more expensive because of increased power consumption and increased wear of the wagons. Velocity is reduced and traffic interruptions or delays during winter must be factored in. A base tunnel is much longer and, therefore, much more expensive and difficult to construct. But it offers many operational advantages.

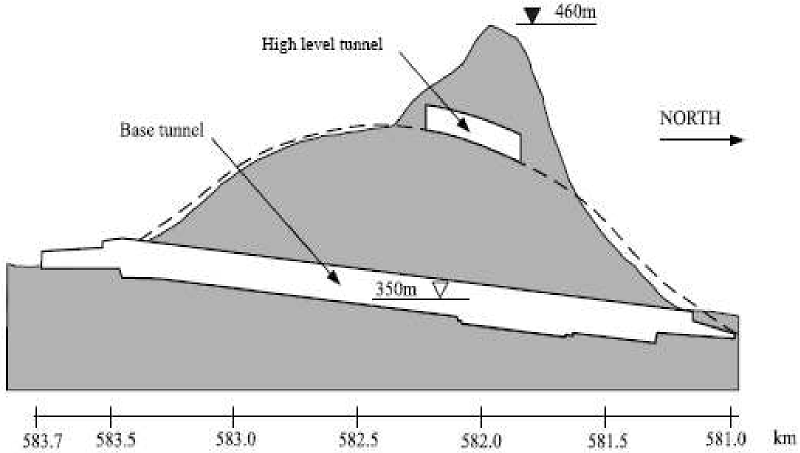


Figure 4-2: high level vs. base level tunnel

1.3. Size and Shape of Tunnels

The size and shape of a tunnel depend upon the nature and type of ground it passes through and also on whether it is designed to carry a single or a double railway line. The shape of a tunnel should be such that the lining is able to resist the pressures exerted by the unsupported walls of the tunnel excavation.

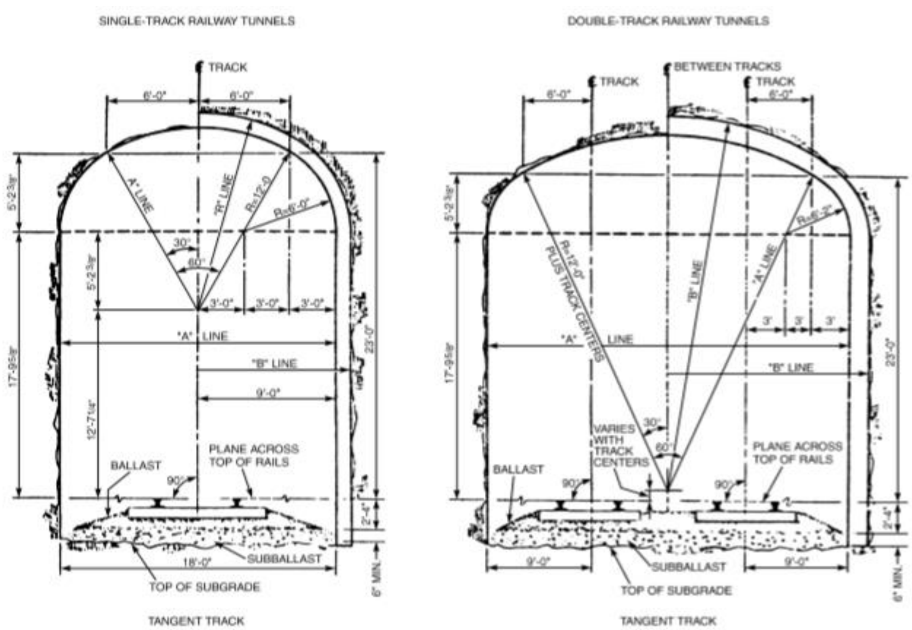


Figure 4-3: Single-Track and Double-Track Railway Tunnels (AREMA)

If the ground is made up of hard rock, then the tunnel can be given any shape. Tunnels in rocky terrains are generally designed with a semicircular arch with vertical sidewalls. In the case of soft ground, such as that consisting of soft clay or sand, the pressure from the sides and the top must be resisted. A circular tunnel is generally best suited for resisting both internal and external forces regardless of the purpose for which the tunnel is used. Theoretically, a circular section provides the largest cross-sectional area for the smallest diameter, which provides greater resistance to external pressure. However, this type of cross section is more useful for drains carrying sewage and fluids and for aqueducts built for irrigation purposes.

For railway track, the circular portion at the bottom of the tunnel has to be leveled in order to lay the track and facilitate the easy removal of muck and placing of concrete. The size of a railway tunnel depends upon the gauge of the railway track and the number of lines.

1.4. Methods of Tunneling

There are various methods of tunneling. The selection of a method depends upon the size of the bore, the condition of the ground, the equipment available, and the extent to which timbering is required. Tunneling may be basically divided into two main groups.

I. Tunneling in hard rocks
II. Tunneling in soft rocks

The various needs for tunneling and the development of technology led to today's method of tunneling. These include the following.

A. Cut and cover method of tunneling
B. Shield tunneling
C. Tunneling using drill and blast systems
D. Use of tunnel boring machines
E. New Austrian Tunneling Method (NATM)

1.4.1. Cut and Cover

This is a simple method of construction for shallow tunnels where a trench is excavated and covered with an overhead support system strong enough to carry the load of what is to be built above the tunnel.

Two basic forms of cut-and-cover tunneling are available:

a) Top-down method

- Here side support walls and capping beams are constructed from ground level by such methods as slurry walling, or contiguous bored piling.
- Then a shallow excavation aids in making the tunnel roof of precast beams or in-situ concrete.
- The surface is then reinstated except for access openings. This allows early reinstatement of roadways, services, and other surface features.
- Excavation then takes place under the permanent tunnel roof, and the base slab is constructed.

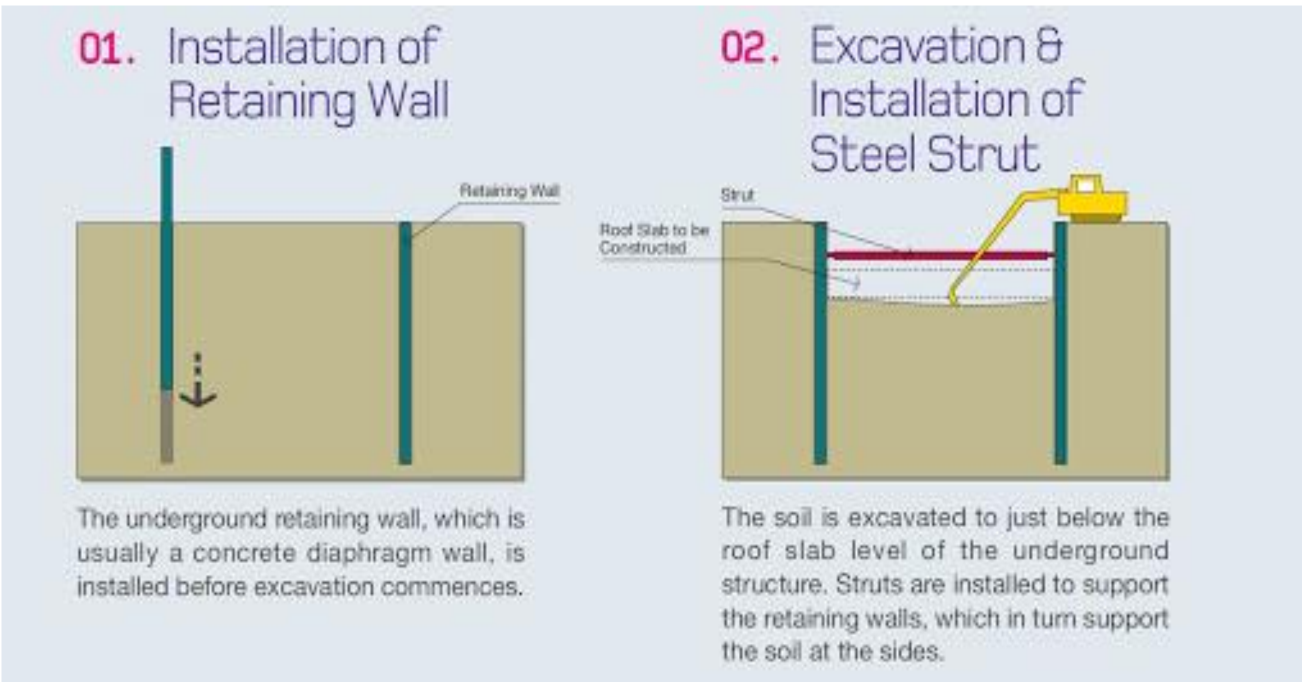


Figure 4-4: ***Top-down method***

b) Bottom-up method

- A trench is excavated, with ground support as necessary, and the tunnel is constructed in it.
- The tunnel may be of in-situ concrete, precast concrete, precast arches, or corrugated steel arches; earlier brickwork was used.
- The trench is then carefully back-filled and the surface is reinstated.

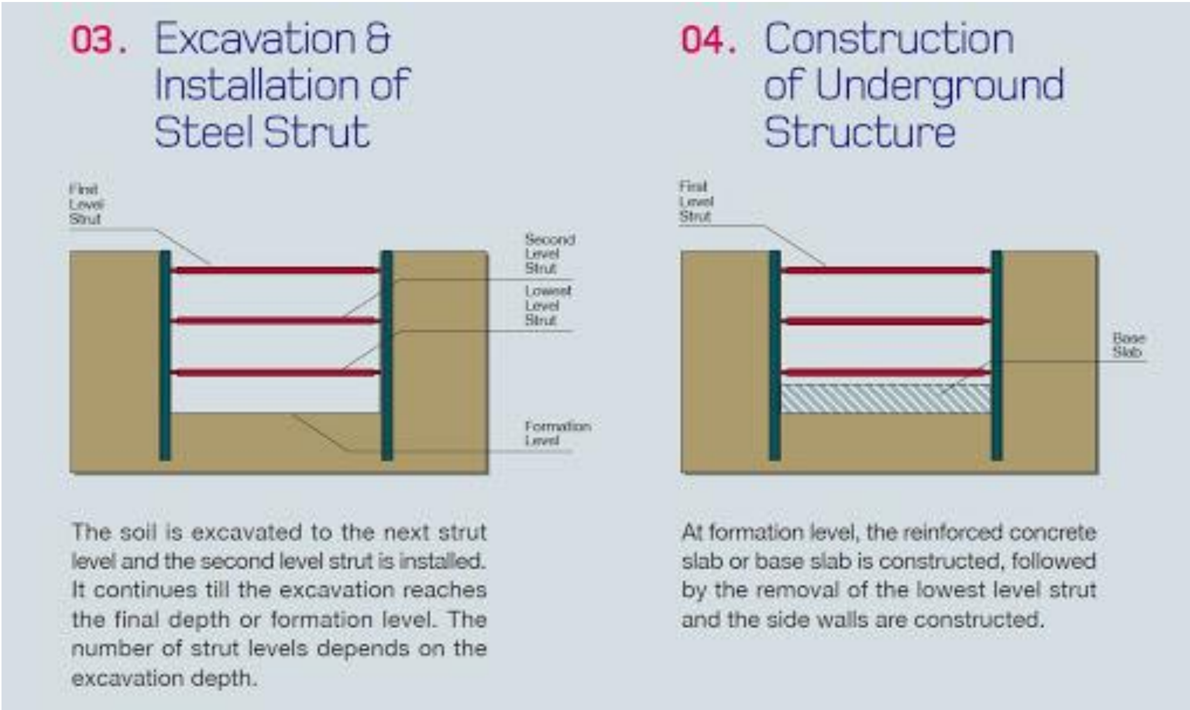


Figure 4-5: ***Bottom-up method***

1.4.2. Shield Tunneling

In soft soil and where deep tunnels are excavated, a tunneling shield is normally adopted. In early shield tunneling, the shield functioned:

- As a way to protect labourers who performed the digging, and moved the shield forward, progressively replacing it with prebuilt sections of a tunnel wall.
- Later shields were used for preventing slippage of earth from the sites into the excavated trench till the tunnel is constructed.
- The shield can then be moved forward to the next section to be taken up.
- The shield also can divide the workface into overlapping portions that each worker could excavate

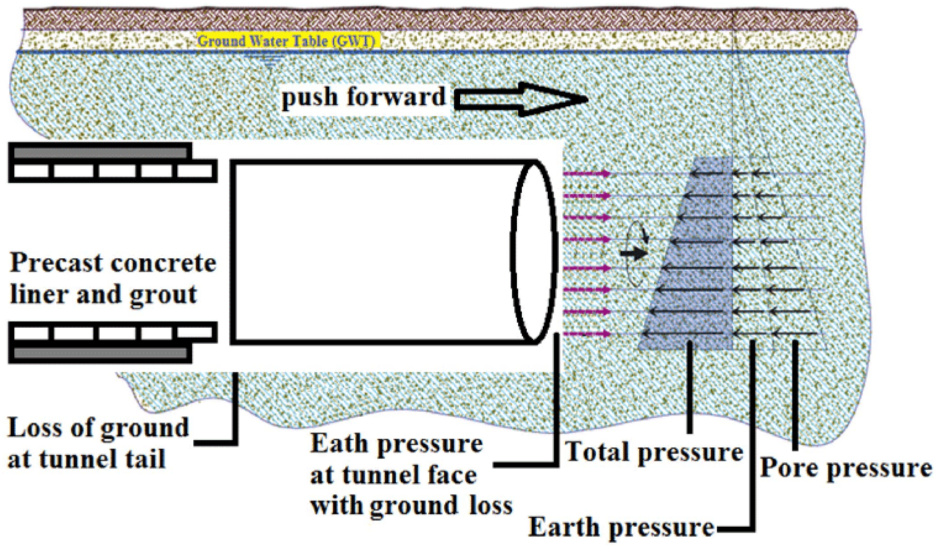


Figure 4-6: Voids in shield tunneling construction

1.4.3. Drill and blast method

This method is used in hard rock. It can be used for full face as well as for header excavation. Drilling jumbos go to the tunnel face and drill a set of holes in the portion to be tunneled. These holes are then charged with controlled explosives and simultaneously blasted. There after special loaders excavate the spoils and the work proceeds cyclically ahead. This method can also be followed in soft soil tunnels, with supports and the portal being fixed at convenient points ahead of the tunneling.

Generally, the drill and blast process is a cyclic operation. Each round consists of four successive operations: drill, blast, muck, and installation of primary support. Drill and blast methods offer the following advantages:

- Fast mobilization and demobilization
- Fastest advance in hard ground
- Lowest cost per m3 of excavation
- Suitable for all excavation shapes
- Flexibility in varying geometry and geology

Figure 4-7: Drill-and-Blast Method

1.4.4. Tunnel boring machines (TBMs)

Tunnel boring machines are used as an alternative to drilling and blasting methods in rock and conventional 'hand mining' in soil.

TBMs have the advantages of minimizing the disturbance to the surrounding ground and producing a smooth tunnel wall. This implies;

- significantly reduces the cost of lining the tunnel, and
- Makes them suitable to use in heavily urbanized areas.

The major disadvantage is the upfront cost. TBMs are expensive and can be difficult to transport. However, as modern tunnels become longer, tunneling with TMBs is much more efficient and results in a shorter project completion time.

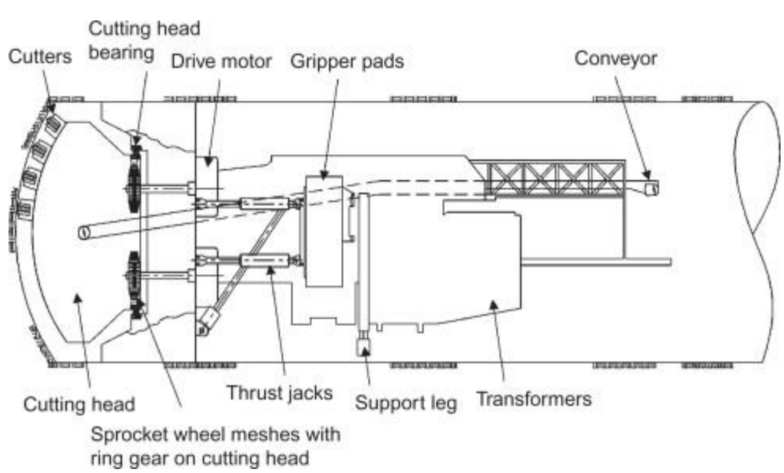


Figure 4-8: Tunnel Boring Machine

Two types of tunnel-boring machines are mostly in use

a. ***Open-face boring*** - suitable for stable soils.
b. ***closed-face shielded machines-*** suitable for less stable soils

Important properties in the excavation processes using TBM

a) ***excavation rate:*** dependent on soil conditions and TBM horsepower
b) ***Stroke length:*** determines how often the TBM will need to be reset.

The tunnel constructions using TBM operations occurred as follows:

1) Excavation and support of the undercut area
2) Excavation of the tunnel and tail tunnel
3) Disposal of dirt from the tunnel face
4) Hoisting dirt to ground level
5) Lining the tunnel
6) Extending the services and rail tracks
7) Excavation and support of the removal shaft.

1.4.5. New Austrian Tunneling method (NATM)

It was developed between 1957 and 1965 in Austria. The main idea is to use the geological stress of the surrounding rock mass to stabilize the tunnel itself. The main features are:

a) mobilization of the strength of rock mass, and
b) Achieving shotcrete protection by applying a thin layer of shotcrete immediately after face advance measurements.

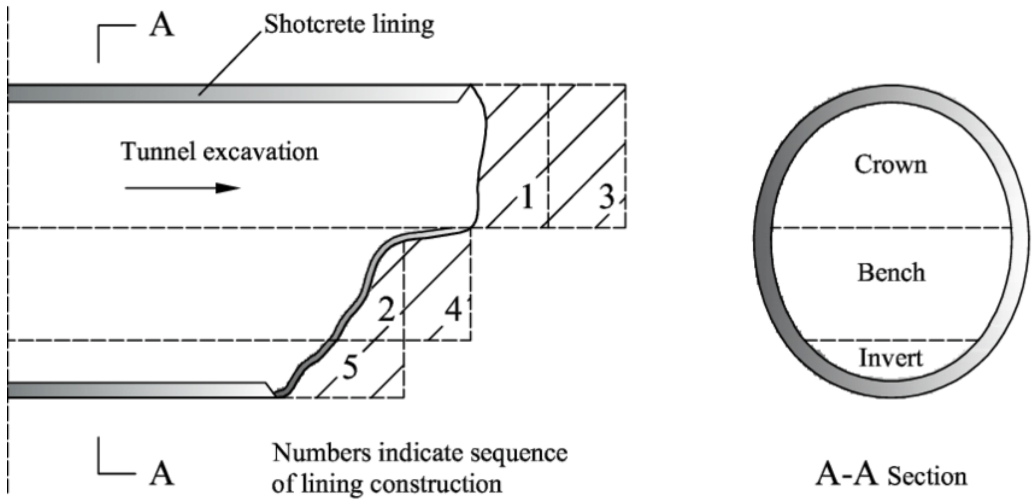


Figure 4-9: New Austrian Tunnelling Method (NATM)

Every deformation of the excavation must be measured, providing flexible support with a primary lining that is thin and reflects recent strata conditions. The tunnel is strengthened by a flexible combination of rock bolts, wire mesh and steel ribs, and quickly closing the invert and creating a load-bearing ring is important. This is crucial in soft ground tunnels where no section of the tunnel should be left open temporarily.

1.5. Selection of Tunnel Excavation Method

The selection depends on various factors, such as:

a. Geological conditions
b. Cross Sectional area and Shape of tunnel
c. Length of tunnel
d. Ground water condition and expected water inflow
e. Vibration restrictions
f. Allowable ground settlements
g. Availability of resources (machinery/equipment, funds & time)

The impact of these factors is uncertain in the design phase; appropriate optimization model shall be employed towards achieving success of a given project. There are various techniques for evaluating risks such as: Monte Carlo Simulation, Event Trees, Fuzzy set, game theory, multicriteria verbal analysis, etc.

1.5.1. Analytical Hierarchy Process (AHP)

Construction management decisions typically involve several conflicting aspects that need to be considered, particularly in tunneling construction projects, known as complex and dynamic projects. Making decisions for projects with such situations can be formulated as multi-criteria optimization problems, where the different aspects of a tunneling project equipment selection constitute the conflicting criteria that are optimized simultaneously.

Analytical Hierarchy Process (AHP) is a very commonly-used tool for multi-criteria analysis decision making. Analytical Hierarchy Process provides methods of weighing selection criteria with a higher level of objectivity, as items are compared two or more at a time.

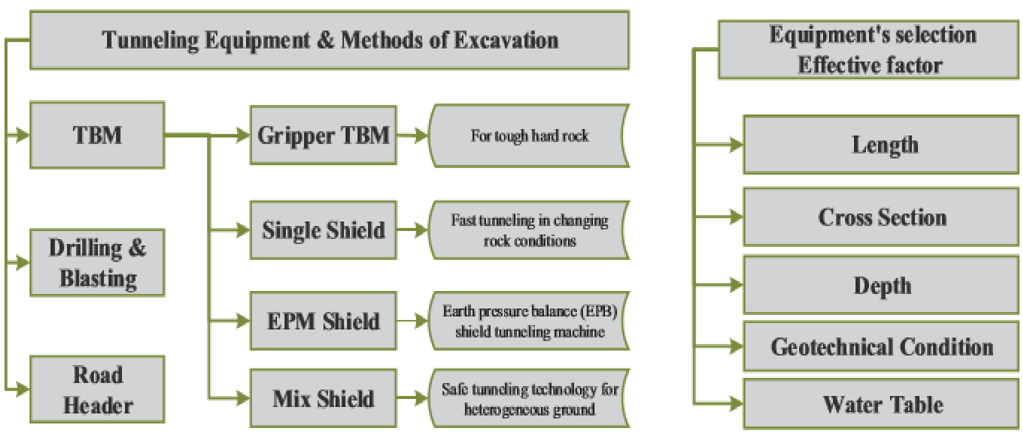


Figure 4-10: Multi Attributed Selection of Excavation Methods in Tunneling Construction (Masouleh, 2015)

Alternatives with respect to project conditions according to Masouleh (2015) is presented as follows:

1. Tunnel Length: divided into 3 main categories:

- short tunnels- L<3000m,
- intermediate tunnel- between 3000m to 6000m,
- long tunnels- a length of 6000m and more

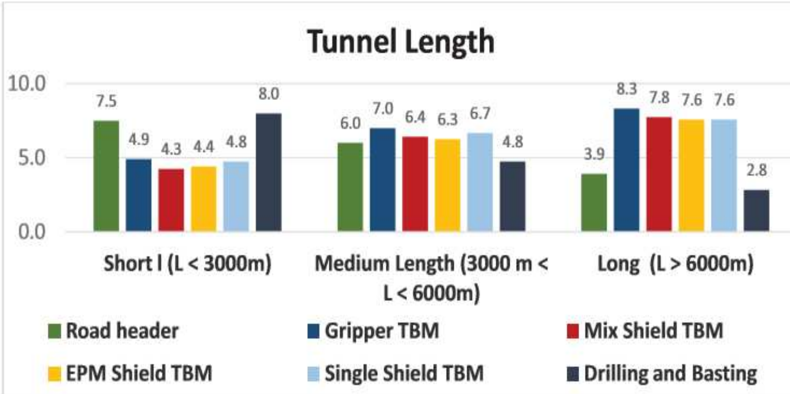


Figure 4-11: Suitability of Equipment with Respect to Tunnel Length

2. Tunnel Cross Section: divided into three main groups:

- Narrow opening tunneling-(R<5m),
- average opening size-(5m<R<12m),
- large opening (R>12m).

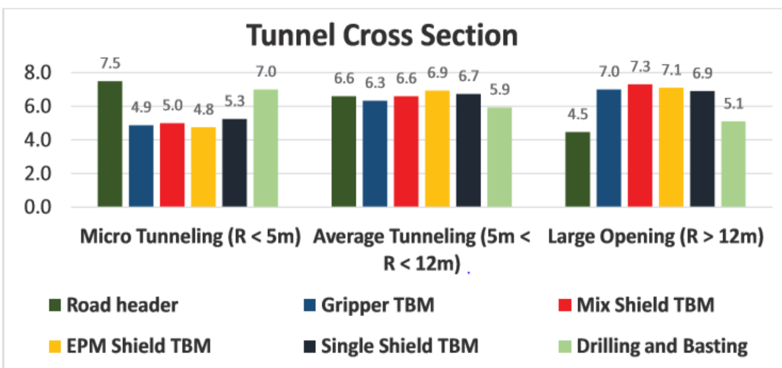


Figure 4-12: Suitability of Equipment with Respect to Tunnel Cross Section

3. Tunnel Depth: three main categories have been defined:

- very deep (D>200m),
- average depth (20m<D<200m), and
- low depth tunnels (D< 20m under the ground level).

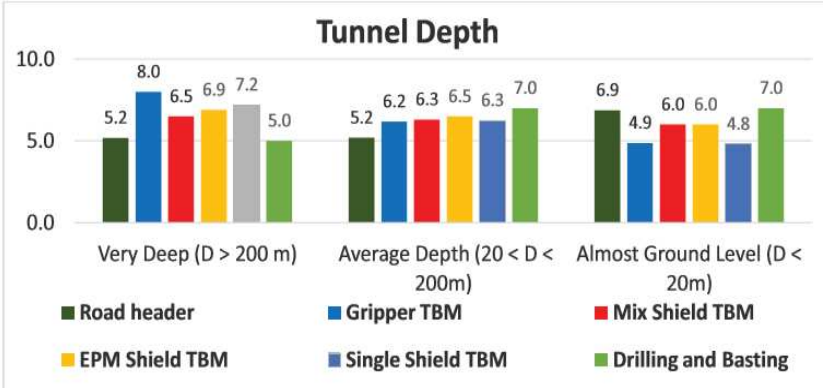


Figure 4-13: Suitability of Equipment with Respect to Tunnel Depth

4. Geotechnical Conditions: six main categories

- sedimentary rock,
- igneous rock,
- metamorphic rock,
- sand & gravel,
- cohesive soil, and
- highly organic soils.

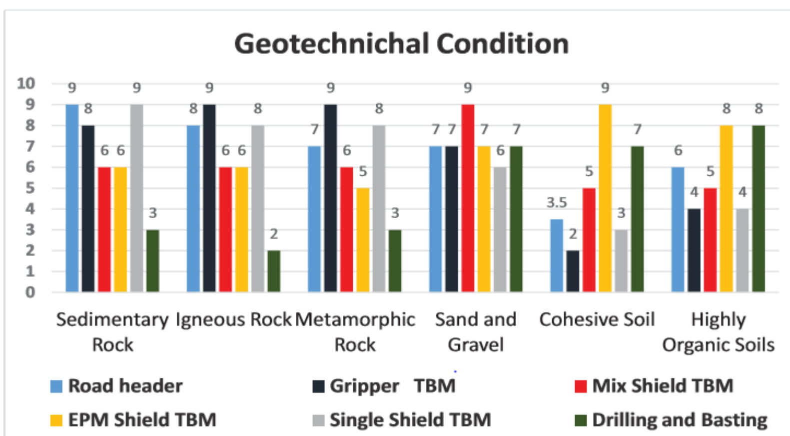


Figure 4-14: Suitability of Equipment with Respect to Geotechnical Conditions

5. Water Table Level: grouped in three categories:

- above the water table,
- partially submerged, and
- Fully submerged in water.

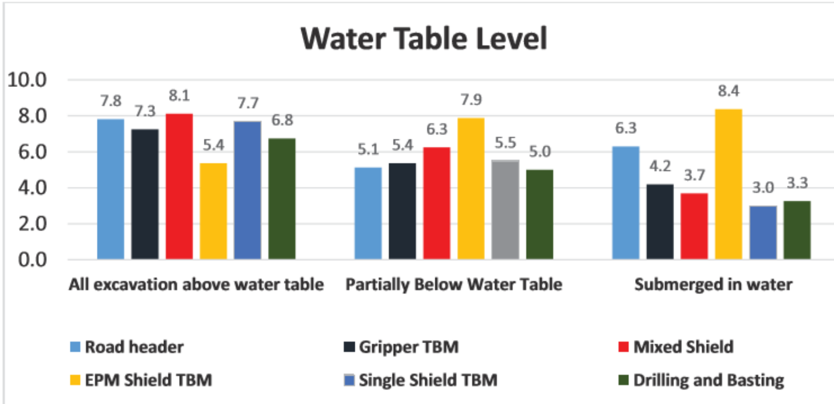


Figure 4-15: Suitability of Equipment with Respect to Water Table Level

The weight result in AHP method represents the relative importance of factors in Figure below, indicating that geotechnical conditions have the greatest influence in tunneling equipment selection with the weight of 0.30.

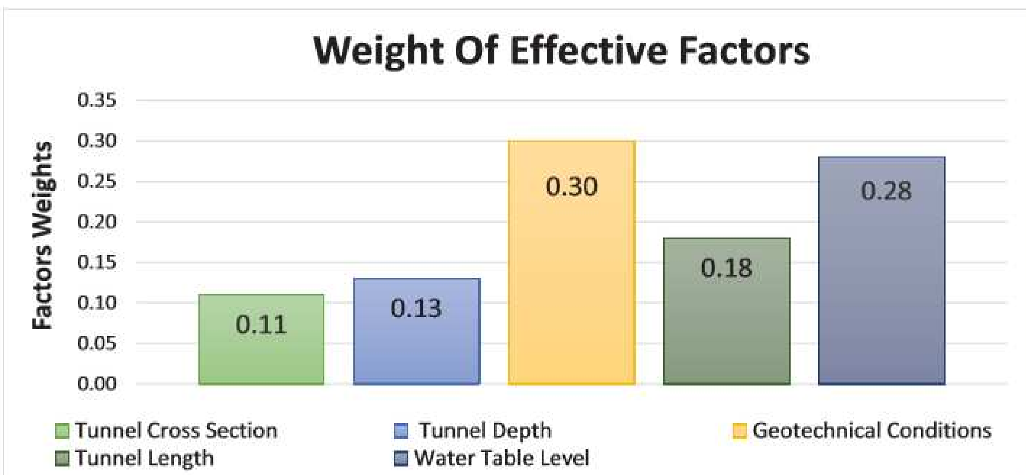


Figure 4-16: Weight Effective Factors in AHP (Masouleh, 2015)

The AHP model is applied as follows:

i. For instance, if a project is a short tunnel, the scores of all six alternatives with respect to this condition, obtained from the survey, are multiplied 0.18 (weight of the tunnel length factor).
ii. The weighted sums of all the scores are calculated for each alternative.
iii. The alternative with the highest overall score is recommended as the most suitable for that particular project.
iv. Moreover, there should be a way to eliminate from the selection the equipment that is not practically based on one or more technical feasibility.
v. For example, if the project is located in a submerged situation, the single shield TBM would be eliminated, even though it might have the highest overall score.

Example 4-1: *select best-fitted tunneling equipment for the project described below.*

PROJECT DESCRIPTION

- *The total length of the tunnel is 5.2 km (average tunneling length)*
- *The tunneling excavation zone starts 20m below ground and under the river bed it reaches 400m below ground in the major part of the project.*
- *The tunnel cross section is R=7.5m,*
- *The tunnel is below the undergrad water table level.*
- *The geotechnical conditions vary, but the overall condition is in the hard rock group, specifically in the sedimentary rock category.*

Solution:

Scenario-1: excavation starts on one end of the tunnel and continues in one direction to reach the other end

- *The weight of each factor is multiplied by each equipment's score*
- *By summing the influence of all factors on every alternative, the total suitability rating of each alternative is determined.*

For road header

1. *Length(average)*
   - *Score =6 out of 9*
   - *weight factor=0.18*
   - *influence of suitablity=6*0.18=<u>1.08</u>*
2. *Cross section (average)*
   - *Score =6.6 out of 9*
   - *weight factor=0.11*
   - *influence of suitability=6.6*0.11=<u>0.726</u>*
3. *geotechnical(sedimentary)*
   - *Score =9 out of 9*
   - *weight factor=0.3*
   - *influence of suitablity=9*0.3=<u>2.7</u>*
4. *depth(very deep)*
   - *Score =5.2 out of 9*
   - *weight factor=0.13*
   - *influence of suitability=5.2*0.13=<u>0.676</u>*
5. *Water level(below)*
   - *Score =6.3 out of 9*
   - *weight factor=0.28*
   - *influence of suitability=6.3*0.28=<u>1,764</u>*

Total suitability rating *=1.08+0.726+2.7+0.676+1.764=**<u>6.946</u>***

*By applying the same procedure for other alternatives, road header with the highest rating of **<u>6.946</u>** is recommended as the most suitable method.*

Scenario 2: it is assumed that excavation is started from two ends simultaneously, using two pieces of equipment of the same type.

- *In this scenario, the project is divided into two 2.6km sections.*
- *This time, in applying the AHP model, the project is considered to be of the short length.*

1.6. Ventilation of Tunnels

A tunnel should be properly ventilated during as well as after construction for the following reasons:

- To provide fresh air to the workers during construction
- To remove the dust created by drilling, blasting, and other tunneling
- To remove dynamite fumes and other objectionable gases produced by the use of dynamites and explosives.

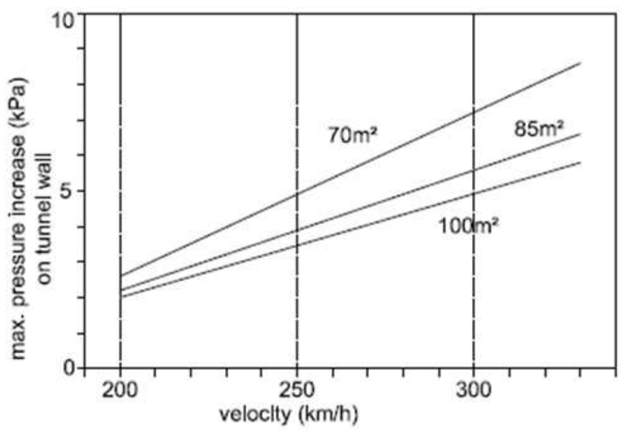


Figure 4-17: aerodynamic pressure rise on the tunnel wall due to entrance of high speed trains

The common methods for the ventilation of a tunnel:

I. *Natural method of ventilation*

This is achieved by drilling a drift through the tunnel from portal to portal. In most cases natural ventilation is not sufficient and artificial ventilation is still required.

II. *Mechanical ventilation by blow-in method*

In the blow-in method, fresh air is forced through a pipe or fabric duct by the means of a fan and supplied near the washing face (or the drilling face; the drilling operation requires the washing of bore holes too). This method has the advantage that fresh air supply is guaranteed where it is required the most. The disadvantage is that the foul air and fumes have to travel a long distance before they can exit the tunnel and in the process it is possible that the incoming fresh air will absorb some dust and smoke particles.

III. Mechanical ventilation by exhaust method

In the exhaust or blow-out method, foul air and fumes are pulled out through a pipe and expelled by a fan. This sets up an air current that facilitates the entrance of fresh air into the tunnel. This method has the advantage that foul air is kept out of the washing face. The disadvantage, however, is that fresh air has to travel a long distance before it can reach the washing face during which period it may absorb some heat and moisture.

IV. Combination of blow-in and blow-out methods

By combining the blow-in and blow-out methods using a blower and an exhaust system, respectively, a tunnel can be provided with the best ventilation. After blasting the ground, the exhaust system is used to remove the smoke and dust. After some time, fresh air is blown in through the ducts and the rotation of the fans is reversed in order to reverse the flow of air.

1.7. Lighting of Tunnels

It is very important to ensure that the tunnels are well lit so that the various activities and operations involved in tunneling can be carried out effectively and safely. The common types of lighting equipment normally used in tunnels are electric lights, coal gas or acetylene gas lights, or lanterns. Electric lights are considered the best option, as these radiate bright light of the required intensity, are free from smoke, etc.

Places where plenty of light should normally be provided are operation points, equipment stations, bottom of shafts, storage points, tempering stations, underground repair shops, etc.

1.8. Drainage of Tunnels

Good drainage of the tunnels is very essential in order for them to operate safely and smoothly during the construction period as well as afterwards. The sources of water for this purpose include groundwater and water collected from the washing of bore holes. Water seeping in up through the ground as well as from the washing of bore holes is collected in sump wells and pumped out. If the tunnel is long, a number of sump wells are provided for the collection of water.

After the construction is over, drainage ditches are provided along the length of the portion of the tunnel that slop from the portal towards the sump well and are used for pumping the water out.

1.9. Shaft of Tunnels

Shafts are vertical wells or passages constructed along the alignment of a tunnel at one or more points between the two entrances for the following purposes:

a. ***Working shafts:*** are generally vertical and provided for the expeditious construction of tunnels by tackling the same at a number of points.
b. ***Ventilation purposes:*** are generally inclined and provided to ensure better ventilation,

1.10. Lining of Tunnels

Except where tunnels pass through sound rock which is expected to be self-supporting, most tunnels are provided with a permanent lining. When tunnelling in relatively soft ground, the lining can be constructed in masonry, brickwork, cast iron, precast concrete or in-situ concrete.

1.10.1. Sequence of Lining

The lining of a tunnel is carried out in the following steps;

Step-1: Guniting

- It is the dry mix shotcrete which convey dry material from a machine to surface of application through a nozzle by means of compressed pressure and high velocity.
- The application is facilitatrd by the addition of water at the nozzle area.
- The mix used in guniting is usually cement mortar mix
- It is done to seal the water in rock tunnels.

Step-2: Concrete lining

- It is done either in one attempt as in the case of circular tunnels or by separately tackling the vest, the sidewall, and the arch.
- For small tunnels that measure 1.2 to 3.0 m in diameter, the concrete lining can be provided by the hand placing method.
- In the case of bigger tunnels, concrete pumps or pneumatic placers are used for placing the concrete.

Step-3: concrete curing

- It is to achieve its maximum strength.
- If the humidity inside the tunnel is not sufficient, curing can be done by spraying water through perforated pipes.

1.10.2. Types and Thickness of Lining

Theoretically, the lining provided inside tunnels may be of timber, iron, steel, brick, or any other construction material but in practical terms the lining provided most commonly is that of reinforced concrete or concrete surface. Concrete lining is provided in tunnels because of:

- its superiority in structural strength,
- ease of placement,
- its durability, and
- lower maintenance cost

The thickness of concrete lining depends upon various factors such as;

- conditions of the ground,
- size and shape of the tunnel,
- soil pressure, and
- the method of concreting

1.10.3. Design of Steel Tunnel Liner Plates

The supporting capacity of a nonrigid tunnel lining such as a steel liner plate results from its ability to deflect under load so that side restraint developed by the lateral resistance of the soil constrains further deflection. Deflection thus tends to equalize radial pressures and to load the tunnel liner as a compression ring.

The load to be carried by the tunnel liner is a function of the type of soil. In a granular soil, with little or no cohesion, the load is a function of the angle of internal friction of the soil and the diameter of the tunnel being constructed. In cohesive soils such as clays and silty clays the load to be carried by the tunnel liner is affected by the shearing strength of the soil above the roof of the tunnel.

A subsurface exploration program and appropriate soil tests should be performed at each installation before undertaking a design.

Here is presented, procedures of steel liner plate design according to AREMA (2010)

Step-1: loads

$$P = P_1 + P_d \tag{4-1}$$

$$P_d = C_d W D \tag{4-2}$$

Where:

- *P = the external pressure on the tunnel liner*
- *P_1 = the vertical pressure at the level of the top of the tunnel liner due to live loads (table below)*
- *P_d = the vertical pressure at the level of the top of the tunnel liner due to dead load*
- *W = total (moist) unit weight of soil*
- *D = horizontal diameter or span*
- *C_d = coefficient for tunnel liner (Figure below)*

Table 4-1: Live Loads, Including Impact, for Various Heights of Cover for Cooper E 80

Height of Cover (Ft)	Load Lb/Ft^2
2	3800
5	2400
8	1600
10	1100
12	800
15	600
20	300
30	100
Note 1: If height of cover (from bottom of cross tie to top of structure) is over 30 feet, use dead load only. For live load other than Cooper E 80, the above values should be accordingly adjusted.	

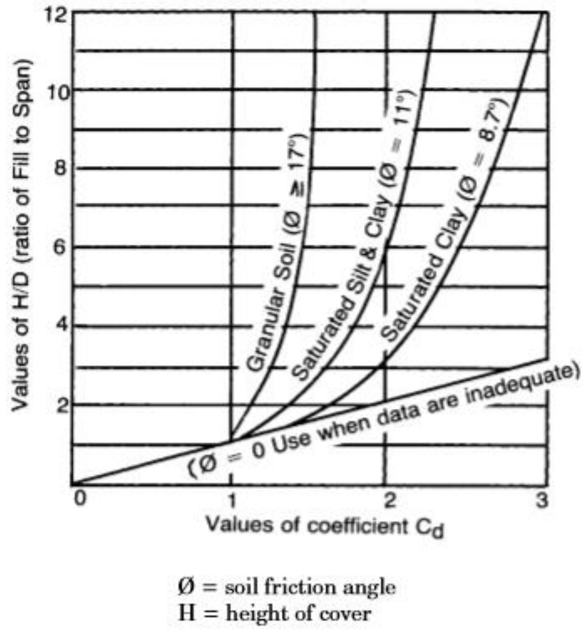


Figure 4-18: Diagram for Coefficient Cd for Tunnels in Soil

Note: If the grouting pressure is greater than the computed external load, the external load P on the tunnel liner shall be the grouting pressure.

Step-2: joint strength

Seam strength for liner plates must be sufficient to withstand the thrust developed from the total load supported by the liner plate. This thrust, T, in pounds per lineal foot is:

$$T = \frac{PD}{2} \quad (4\text{-}3)$$

$$T * safety\ factor \leq ultimate\ strength \quad (4\text{-}4)$$

Where: P = the external pressure on the tunnel liner; D = diameter or span in feet; safety factor (longitudinal seam strength=3)

Table 4-2: Longitudinal Seam Strengths

Plate Thickness Inches	Ultimate Strength, Kips/Ft	
	2 flange	4 flange
0.075	20.0	–
0.105	30.0	26.4
0.135	47.0	43.5
0.164	55.0	50.2
0.179	62.0	54.5
0.209	87.0	67.1
0.239	92.0	81.5
0.250	–	84.1
0.313	–	115.1
0.375	–	119.1

Step-3: Minimum Stiffness for Installation

The liner plate ring shall have enough rigidity to resist the unbalanced loadings of normal construction: grouting pressure, local slough-ins and miscellaneous concentrated loads. The minimum stiffness required for these loads can be expressed for convenience by the formula below.

4-5

$$\min imum\, stiffness = \frac{EI}{D^2}$$

Where: D = diameter in inches E = modulus of elasticity, psi I = moment of inertia, in.4/in

$$For\, 2 - flange\, \frac{EI}{D^2} = 50 \min imum$$

$$For\, 4 - flange\, \frac{EI}{D^2} = 110 \min imum$$

Step-4: Critical Buckling of Liner Plate Wall

Wall buckling stresses are determined from the following formulas:

4-6

$$\text{For diameters less than } \frac{r}{k}\sqrt{\frac{24E}{f_u}} : f_c = f_u - \frac{f_u^{\,2}}{48E}\left(\frac{KD}{r}\right)^2 \text{ in psi}$$

4-7

For diameters greater than $\frac{r}{k}\sqrt{\frac{24E}{f_u}}$: $f_c = \frac{12E}{\left(\frac{KD}{r}\right)^2}$ in psi

Where:

- f_u = *minimum specified tensile strength, psi*
- f_c = *buckling stress, psi, not to exceed specified yield strength*
- *K = soil stiffness factor, which will vary from 0.22 for soils where Ø* $> 15^o$ *to 0.44 where Ø* $<= 15^o$ *to 0.44*
- *D = pipe diameter, inches*
- *r = radius of gyration of section*
- *E = modulus of elasticity, psi*

Finally, design for buckling is accomplished by limiting the ring compression thrust T;

4-8

$$T = \frac{f_c A}{FS}$$

Where: T = thrust per lineal foot, A = effective cross section area of liner plate, square inches per foot, FS = factor of safety for buckling=2

Table 4-3: Effective Sectional Properties Based on the Average of One Ring of Plates (AREMA, 1992)

Thickness (Inches)	Area (Inch ^{2}/Inch)	Moment of Inertia (Inch ^{4}/Inch)	Radius of Gyration (Inch)
18 inch Wide – 2 Flange Plates			
0.075	0.096	0.034	0.60
0.105	0.135	0.049	0.60
0.135	0.174	0.064	0.61
0.164	0.213	0.079	0.61
0.179	0.233	0.087	0.61
0.209	0.272	0.103	0.62
0.239	0.312	0.118	0.62
16 inch Wide – 4 Flange Plates			
0.105	0.0656	0.0398	0.41
0.120	0.0759	0.0468	0.41
0.135	0.0851	0.0517	0.41
0.164	0.1040	0.0709	0.44
0.179	0.1140	0.0771	0.44
0.209	0.1380	0.0904	0.44
0.239	0.1510	0.1180	0.47
0.313	0.1930	0.1620	0.49
0.375	0.2290	0.2200	0.52

Deflection of a tunnel depends significantly on the amount of over excavation of the bore and is affected by delay in backpacking or inadequate backpacking. The magnitude of deflection is not primarily a function of soil modulus or the liner plate properties, so it cannot be computed with usual deflection formulas.

Where the tunnel clearances are important, the designer should oversize the structure to provide for a normal deflection. Good construction methods should result in deflections of not more than 3% of the nominal diameter.

1.11. Tunnel Inspection and Maintenance

Well-constructed tunnels should require little maintenance, especially if they are relatively dry. However, access needs to be arranged for tunnels to be inspected in detail on a regular basis. The frequency should be based on the general condition of the tunnel, its material of construction and the amount of water present.

Records must be kept of all inspections and repairs and the amount of ingress of water. Brick tunnels require periodic repointing but most concrete and cast iron lined tunnels require little attention.

The following features of a tunnel require detailed inspection and examination.

1. ***Portal at either end:*** This is inspected to ascertain whether there are any signs of slips in the slopes above the portals; or whether the masonry is in any way cracked, shaken, or bulging; or whether there are any apparent signs of movement.
2. ***Section of tunnel in relation to moving dimensions:*** This inspection is done to check if the section, whether on straight routes or curves, conforms to the schedule of dimensions.
3. ***Tunnel walls and roofing:*** These are inspected in the case of lined tunnels; to ascertain whether the lining is in a satisfactory condition. Doubtful places, such as loose projections, should be tapped to check for hollow sound or loose rock.
4. ***Drainage:*** This is inspected to ascertain whether the side drains are adequate and functional
5. ***Ventilation shafts:*** These are inspected to ascertain whether the ventilation shafts are adequate and free from vegetation and other growth.
6. ***Lighting equipment and special tools:*** This inspection is required to ascertain whether the lighting equipment and special tools, wherever supplied, are in a good state.
7. ***Track:*** The track is inspected to ascertain whether its line and level are correct, including that of the approaches. Rails, sleepers, and fastenings should be particularly examined for corrosion.
8. ***Trolley refuges:*** In order to provide safety to the engineers inspecting the tunnel on trolley as well as for the safety of technical staff carrying out work in tunnels, trolley refuges are provided so that there can be safe places, where trolley or technical staff can wait, when trains pass.

1.12. Safety in Tunnel Construction

In all civil engineering construction safety is an essential ingredient. In tunneling this is an absolutely primary consideration and must never be allowed to become secondary to considerations of cost or speed of construction.

Tunneling is a difficult, hazardous, and time-consuming process and the whole operation has to be done systematically so that safety is ensured at all times. Normally accidents in a tunnel occur under the following circumstances.

a) Falling rocks
b) During the loading and hauling of muck
c) Poor handling of explosives
d) During shaft operations
e) Cramped working space

Therefore, appropriate provisions must be applied for preventing accidents during tunneling operations.

EXERCISE 4-1:

1. What are alternative scenarios to be compared with a tunnel, in time of decision making, in both urban and regional railway line?
2. Which considerations make tunnel alignment and gradient design in different with the rest of railway line? And why?
3. Which items in the lifetime cost (LTC) of railway line are more sensitive to a tunnel being a base level; or high level?
4. Suppose the major part of a tunnel 'A' contains sand, silt, and clay with total project length of 3.6 km and tunnel cross section of 6.1m. The tunnel is above ground water level and excavation zone starts 80m below ground. Which method of tunneling do you recommend? And why?
5. Which factors are more important in selecting lining type for a given project? And why?
6. Discuss on facilities for assuring safety in tunneling activity?

MODULE-FIVE

5. RAILWAY BRIDGES

Introduction

Railroad Bridge is a structure constructed for the exclusive purpose of carrying railroad traffic across an obstruction. Since the construction of the first modern railway bridge in the 1880s, the discipline has evolved extensively. Railway bridges are specific structures requiring a deep knowledge not only of structural engineering, but also of the operation and safety requirements of lines used daily by great number of passengers.

1.1. Types of Bridges

Based on traffic carried, bridges can be categorized into following types:

a. Road bridges or Highway bridges
b. Railway or rail bridges
c. Road cum rail bridges (facilitating both rail and road way)

A number of differences exist between railroad and highway bridges:

a) The ratio of live load to dead load is much higher than for a similarly sized highway structure. This can lead to serviceability issues such as fatigue and deflection control governing designs rather than strength.
b) The design impact load on railroad bridges is higher than on highway structures
c) Simple span structures are preferred over continuous structures for railroad bridges. many of the factors that make continuous spans attractive for highway structures are not as advantageous for railway use.
d) Interruptions in services are typically much more critical for railroads than for highway agencies. Therefore, constructability and maintainability without interruption to traffic are crucial for railroad bridges.

e) Since the bridges support the track structure, the combination of track and bridge movement cannot exceed the tolerances in track standards. Interaction between the track and the bridge should be considered in design and detailing.
f) Seismic performance of highway and railway bridges can vary significantly. railroad bridges have performed well
g) Railroad bridge owners typically expect a longer service life from their structures than Highway Bridge owners expect from theirs.

1.1. Bridge Structures and Materials

1.1.1. Reinforced Concrete Structures

Reinforced concrete is usually used in conjunction with welded steelwork or prestressed concrete as part of the bridge deck and often forms the parapets. For Simple components which are not subjected to excessive loads or moments; in the substructure, abutments and wing walls in place of mass concrete, brickwork or masonry

1.1.2. Prestressed Concrete Structures

Although RCC adds steel where tension is likely to occur, it does lead to heavy structures as up to half of the concrete is not contributing to the structural strength.

PSC introduced a precompression into the concrete in areas where tension would otherwise develop under loading, which is at least equal to the applied tension, thus negating that tensile stress. In this way the whole concrete section remains under compression and therefore ‘works’. In general terms there are two methods of introducing prestress into a concrete member: pretensioning and posttensioning.

1.1.3. Steel Structures

For medium spans, steel bridges are usually constructed with single webbed ‘I’ section main girders either under the track or alongside with cross girders.

For longer spans welded steel bridges are often constructed with main girders in the form of a box. For double track railway bridges with much larger spans, in excess of about 30 metres, the main girders can be in the form of a lattice or truss. Painting methods and preparation need to be very carefully considered on all new steel structures.

1.1.3.1. Bridge Decks for Steel Railway Bridges

Table 5-1: Open-Bridge Decks

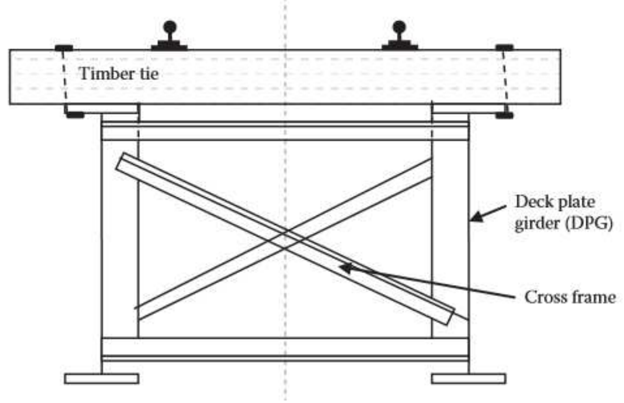
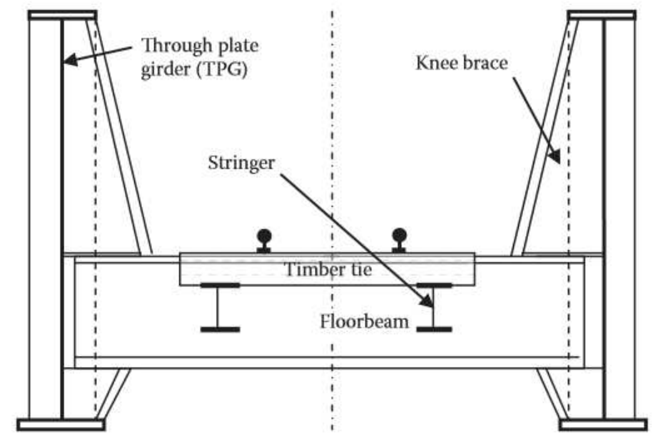
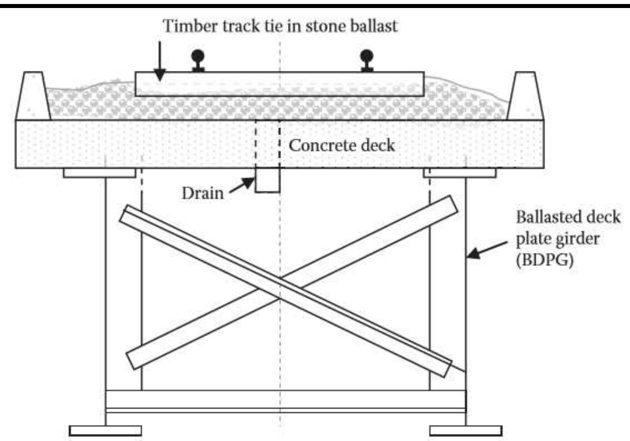
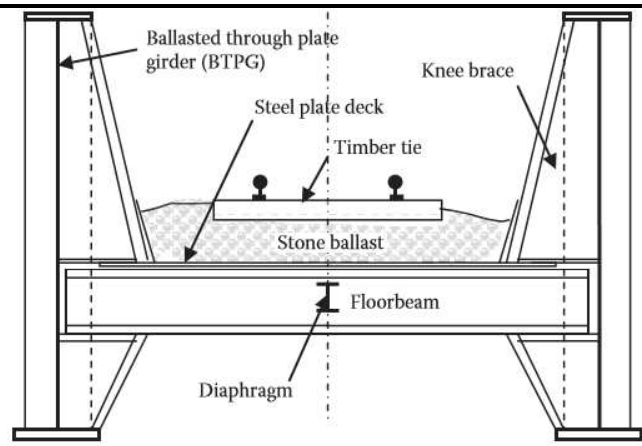
(a) Open-deck plate girder (DPG) span	(b) Open through plate girder (TPG) span
	

Table 5-2: Ballasted Bridge Decks

(a) Ballasted deck plate girder (BDPG) span	(b) Ballasted through plate girder (BTPG) span
	

Bridge Bearings

Spans usually have provision for expansion to accommodate horizontal movements due to temperature or other longitudinal effects. The bearings should also transmit vertical, lateral horizontal, and, in the case of fixed bearings, longitudinal horizontal forces.

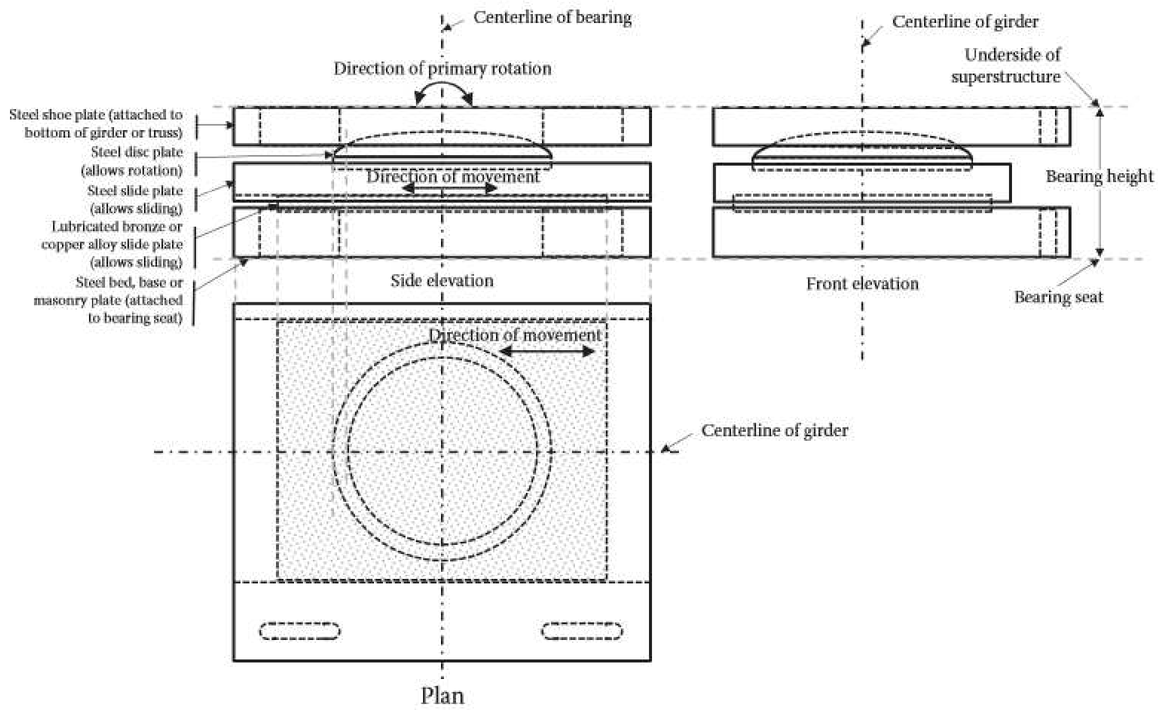


Figure 5-1: Multirotational steel expansion bearing

1.1.4. Brick and Masonry Structures

It is important to keep joints well pointed and any drainage well rodded to ensure that water does not get into the structure.

- In very large masonry piers it is wise to check internal condition by trial bores at different levels. Any voids that are found or softness of material should be pressure grouted to prevent further deterioration.
- Because of high labour costs, it is not usual today to use masonry or brickwork for large gravity structures.
- If appearance is a consideration, structures can be constructed in concrete and then faced in brick or masonry.

1.2. Loads on Railway Bridges

1.2.1. Permanent Loads

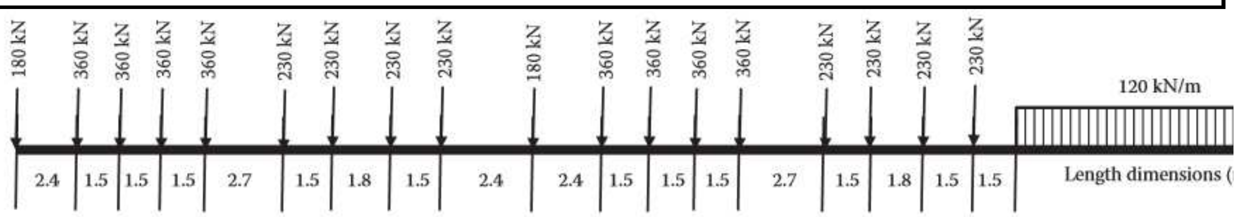
Table 5-3: Dead Loads on Steel Railway Bridges (John, 2018)

Item	Dead Load	
Track (rails and fastenings)	200 lb/ft	300 kg/m
Steel	490 lb/ft³	7850 kg/m³
Reinforced and prestressed concrete	150 lb/ft³	2400 kg/m³
Plain (unreinforced) concrete	145 lb/ft³	2320 kg/m³
Timber	35–60 lb/ft³	560–960 kg/m³
Sand and gravel, compacted (railroad ballast)	120 lb/ft³	1920 kg/m³
Sand and gravel, loose	100 lb/ft³	1600 kg/m²
Permanent formwork (incl. concrete in valleys)	15 lb/ft²	75 kg/m²
Waterproofing on decks	10 lb/ft²	50 kg/m²

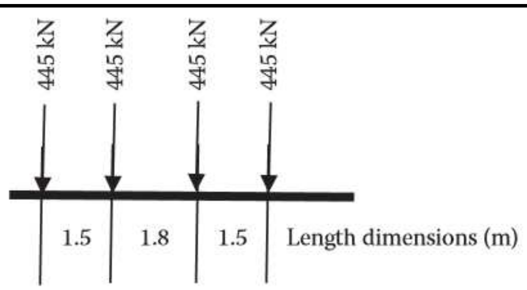
1.2.2. Vehicular Live Load

Table 5-4: Cooper's Axle Loads and alternate live load—axle loads (AREMA, 2015)

(a) Cooper's EM360 live load



(b) Alternate live load



1.2.3. Dynamic Loads

a. **Steel railway bridges** (John, 2018)

5-1

$$I_f = RE + I_v$$

5-2

$$RE = 0.2W$$

Percentage of live load for rolling equipment without hammer blow (freight and passenger cars, and Locomotives other than steam):

5-3

$$\text{For L less than 24m}: 40 - \frac{3L^2}{145}$$

5-4

$$\text{For L 24m or more}: 16 + \frac{183}{L - 9}$$

Where: I_f - impact factor for railway bridge; I_v - the vertical effects due to superstructure – vehicle interaction; RE- the effects due to vehicle rocking; W - the wheel load(1/2 axle load).

Table 5-5: impact load for members receiving load from more than one track

Span Length ,L	Impact
Loads Received From Two Tracks	
For L less than 53m	Full impact on two tracks
For L from 53mto 69m	Full impact on one track and a percentage of Full impact on the other as given by the formula, 450-6.56L
For L greater than 69m	Full impact on one track and none on the other
Load Received From More than Two Tracks	
For all values of L	Full impact on any two tracks that creates the largest load effect

b. Concrete railway bridges

For rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.), the impact shall be equal to the following percentages of the live load:

- *For L<=4 meters; I=60*
- *For 4 meters < L<=39 meters;* $I = \frac{125}{\sqrt{L}}$
- *For L > 39 meters I = 20, L is the span length in meters*

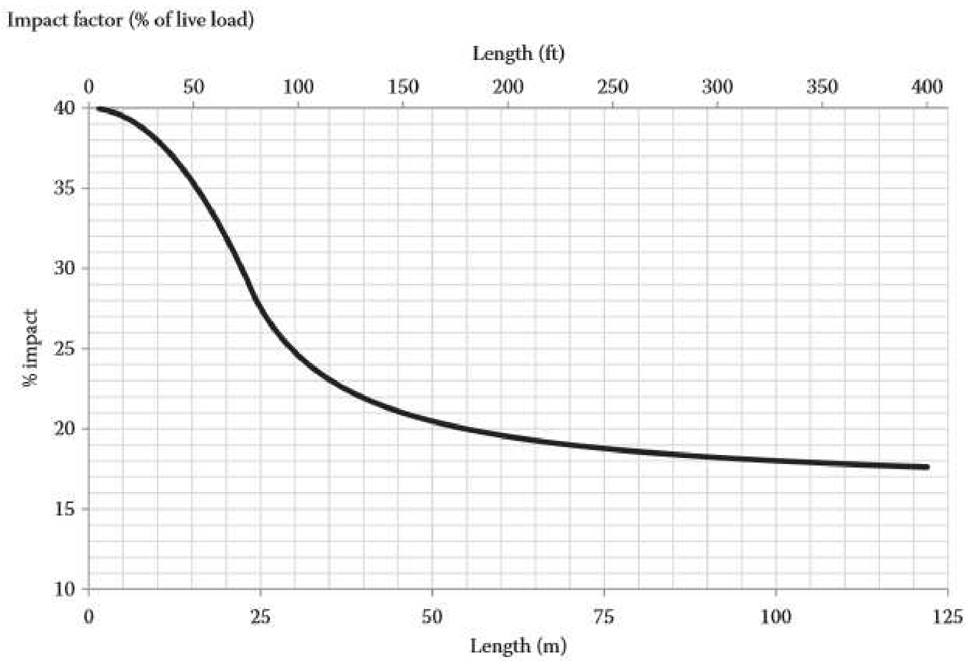


Figure 5-2: AREMA design impact for simply supported spans due to vertical effects as a percentage of live load (diesel locomotives and modern freight cars).

1.2.4. Centrifugal Force

Centrifugal forces acting horizontally at the vehicle center of gravity (recommended as 2.4m above the top of the rails in AREMA, 2010) act on the moving live load as it traverse the curved track on the bridge. The centrifugal force corresponding to each axle load is

$$CF_A = \frac{m_A V^2}{R} \tag{5-5}$$

Where $m_A = A/g$; where A is the axle load ,g is the acceleration due to gravity ,V is the speed of the train in mph(Km/h),and R is the radius of curvature

5-6

$$S = \sqrt{\left(\frac{E+75}{0.0086D}\right)}$$

Where: C = Centrifugal force in percentage of the live load, D = Degree of curve (Degrees based on 30 m) chord, E = Actual super-elevation in mm, S = Permissible speed in km/hr

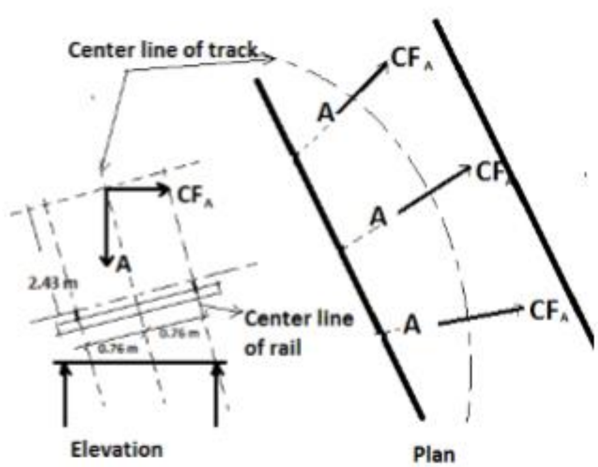


Figure 5-3: Centrifugal forces from a curved track of Railway Bridge

Other Loads significant for Railway Bridge:

- Lateral forces from equipment
- Lateral vibration loads
- Forces from continuously welded rail (CWR)
- Loads for stability check
- Loads for overall lateral stability
- Geometrical load and Resonance phenomena

1.2.5. Longitudinal Forces Due To Braking and Traction

The longitudinal force for EM-360 loading shall be taken as the larger of (John, 2018):

5-7

$$\text{Longitudinal braking force (KN)} = 200 + 17.5L; \text{ acting } 2500\text{mm above top of rail:}$$
$$\text{Longitudinal traction force (kN)} = 200\sqrt{L}\ ; \text{ acting } 900\text{mm above top of rail:}$$

Where: L is length in meters of the portion of the bridge under consideration.

1.2.6. Wind Forces

a. Wind forces on loaded bridge:-includes (AREMA)

- Lateral wind loading on train: shall be taken at 1.33 KN per linear meter applied normal to the train on one track at a distance of 2.4m above top of rail.
- Lateral Wind loading on bridge: shall be taken at 1.48KN/m2
- The lateral wind force on girder and truss spans:
- The longitudinal wind force on spans shall be taken as:

b. Wind forces on unloaded bridge

- Lateral wind loading: shall be taken as 2.5 KN /m2
- Longitudinal wind loading for girders, truss spans, viaduct towers and bents

1.2.7. Distribution of Live Load

Unlike highway loads, which may move laterally across the bridge deck, railway live loads are generally fixed in lateral position. However, they are a longitudinal series of large magnitude concentrated wheel loads, and longitudinal and lateral distribution to the deck and supporting members must be considered.

A. **For open deck bridges:** no longitudinal distribution is made and lateral distribution to supporting members is based on span cross-sectional geometry and type of lateral bracing system.
B. **For ballasted deck bridges:**

Table 5-6: distribution to deck

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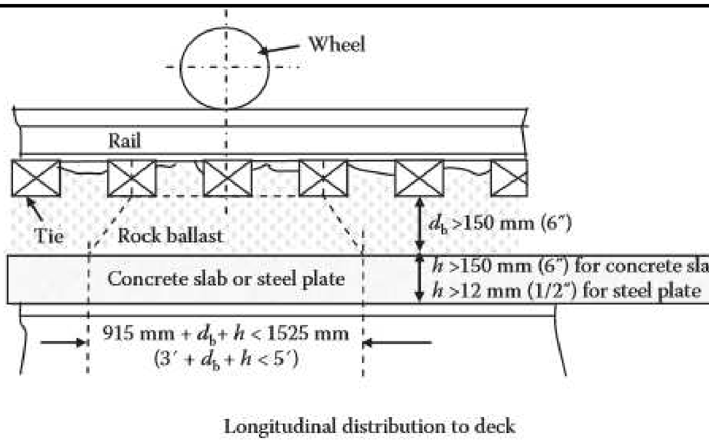
<tr>
<td>
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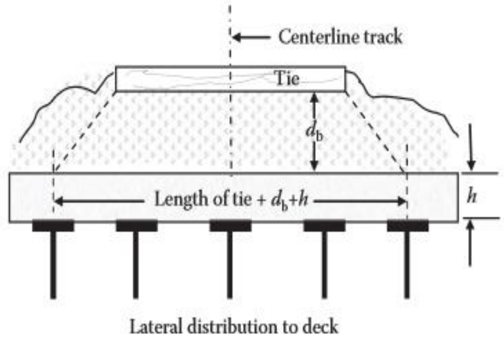
<td>
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Table 5-7: Distribution with transverse floorbeams

<table>
<tr>
<th>Longitudinal distribution</th>
<th>lateral distribution</th>
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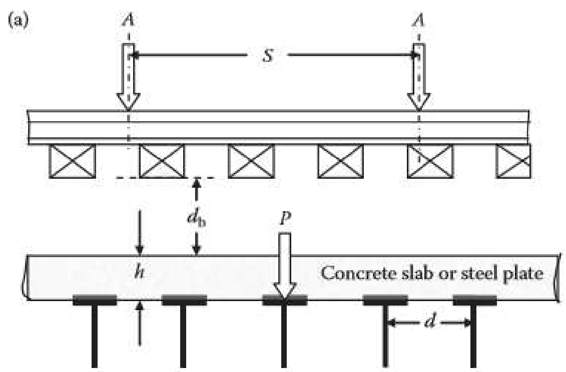
<td>
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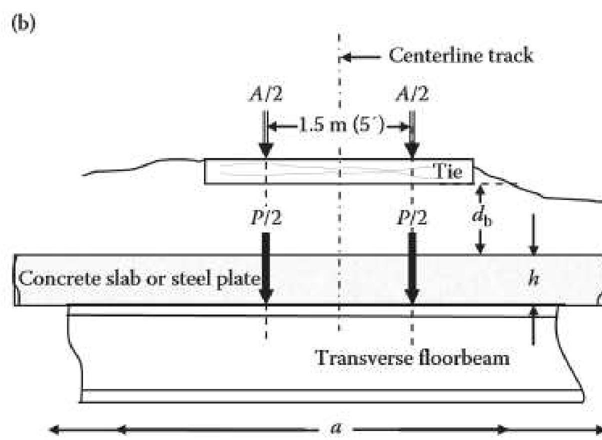
<td>
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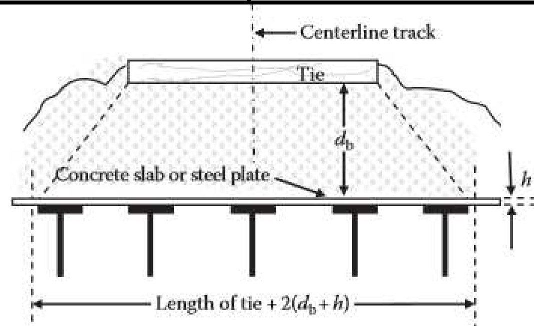


Figure 5-4: Lateral distribution of live load to longitudinal members

1.2.8. Multiple Presences

For members receiving load from more than one track, the design live load on the tracks shall be as follows (AREMA):

- For two tracks, full live load on two tracks.
- For three tracks, full live load on two tracks and one-half on the other track.
- For four tracks, full live load on two tracks, one-half on one track, and one-quarter on the remaining one.
- For more than four tracks, as specified by the Engineer.
- The selection of the tracks for these loads shall be such as will produce the greatest live load stress in the member.

1.3. Fatigue Loads and Serviceability Requirements

1.3.1. Fatigue

The major factors governing fatigue strength at a particular location of a member or connection are: the number of stress cycles, the magnitude of the stress range, and the relevant Fatigue detail category.

Table 5-8: number of stress cycles, N (AREMA)

Member Description	Span Length, L of Flexural Member or Truss or Load Condition	Constant Stress Cycles, N
	Classification I	
Longitudinal flexural members and their connections. Truss chord members including end posts, and their connections	L > 30m	2,000,000
	L ≤30m	> 2,000,000
	Classification II	
Floorbeams and their connections. Truss hangers and sub-diagonals that carry floorbeam reactions only, and their connections. Truss web members and their connections.	Two Tracks Loaded	2,000,000
	One Track Loaded	> 2,000,000

The stress range, SR, is defined as the algebraic difference between the maximum and minimum calculated stress due to dead load, live load, mean impact load, and centrifugal load.

Table 5-9: Stress range (AREMA)

Allowable Fatigue Stress Range for Number of Design Stress Range Cycles >2,000,000

Member or Connection Condition	S_{rfat} (MPa)	S_{rfat} (ksi)
Plain member	165	24
Bolted slip-resistant connection	110	16
Partial penetration groove and fillet welded connection	18–69	2.6–10
Full penetration weld connection	18–110	2.6–16

Table 5-10: Mean Impact Load for fatigue (AREMA)

Member	Percentage
Members with loaded lengths ≤ 3m and no load sharing	65%
Hangers	40%
Other Truss members	65%
Beams ,Stringers ,Girders and Floor Beams	35%

1.3.2. Pedestrian Load

Typical walkways for steel railway bridges consist of a steel grating or other system with nonslip surfaces and designed for maximum deflection requirments with a given load.

1.3.3. Deflection

Live load deflection control is a significant serviceability criterion. Track standards limit the amount of deflection in track under train passage.

1.4. Load Combinations

Table 5-11: Load combinations for steel railway superstructure design (AREMA)

Load Combinations for Steel Railway Superstructure Design

Load Case	Load Combination	Member	F_L
D1-A	DL + LL + I + CF	All members subjected to LL	1.00
D1-B	LL + I + CF	All members with tensile stress ranges from LL	S_{rfat}
D2-A	DL + LL + I + CF + W_L + LF + N + CWR + OF	All members subjected to LL, excluding floorbeam hangers	1.25
D2-B	DL + LL + I + CF + W + LF + N + CWR + OF	Floorbeam hangers	1.00
D3	DL+ LL_T+ I_T+ CF_T	Truss web members and connections	1.33
D4-A	W_L or LV	Members loaded by wind only	1.00
D4-B	W_{UL}	Members loaded by wind only	1.00
D5-A	CF + BF + N + W_L + CWR	Bracing between compression members	1.25
D5-B	DF	Cross frames, diaphragms, and anchor rods	1.50
E1-A	DL + EQ	All members in active seismic zones	1.50
E1-B	DL+ LL + I + CF + EQ	Members in active seismic zones in long bridges only	1.50
S1-A	SL + N + CF	Members resisting overall instability	1.50
S1-B	Q	Members in deck spans resisting overall instability	1.50
C1	DL	Members stressed during lifting or jacking	1.50
C2-A	DL	Members stressed during erection	1.25
C2-B	DL + W_{UL}	Members stressed during erection	1.33

Example 5-1:

Design deck plate and intermediate floor beams of ballasted through plate girder (BTPG) with the following information.

GENERAL INFORMATION

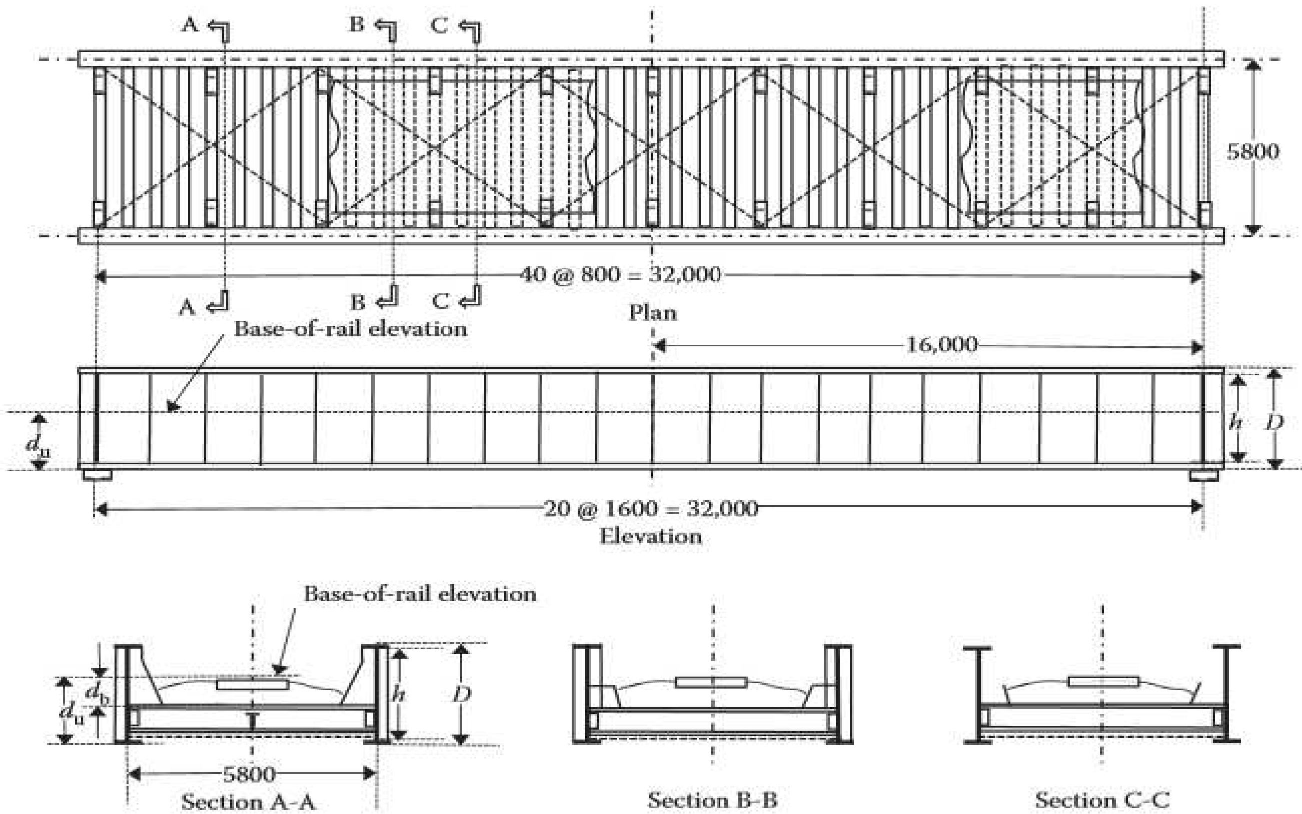
The tangent track on the proposed BTPG span will consist of 180 mm deep × 2500 mm long track ties on 20 mm tie plates with 450 mm ballast from the base of rail to the top of the deck plate with a 32.6 m long BTPG span.

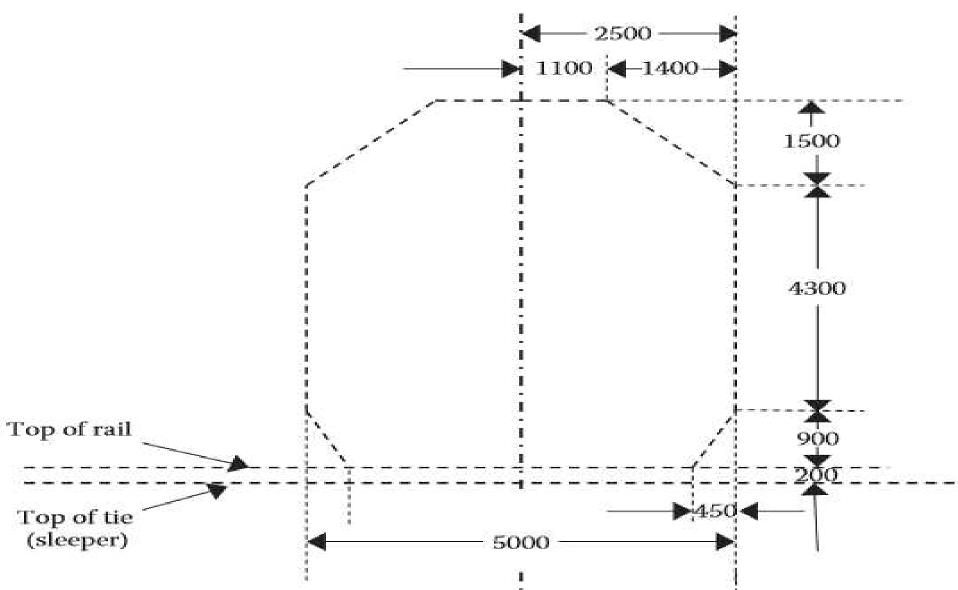
LOADING AND MATERIALS

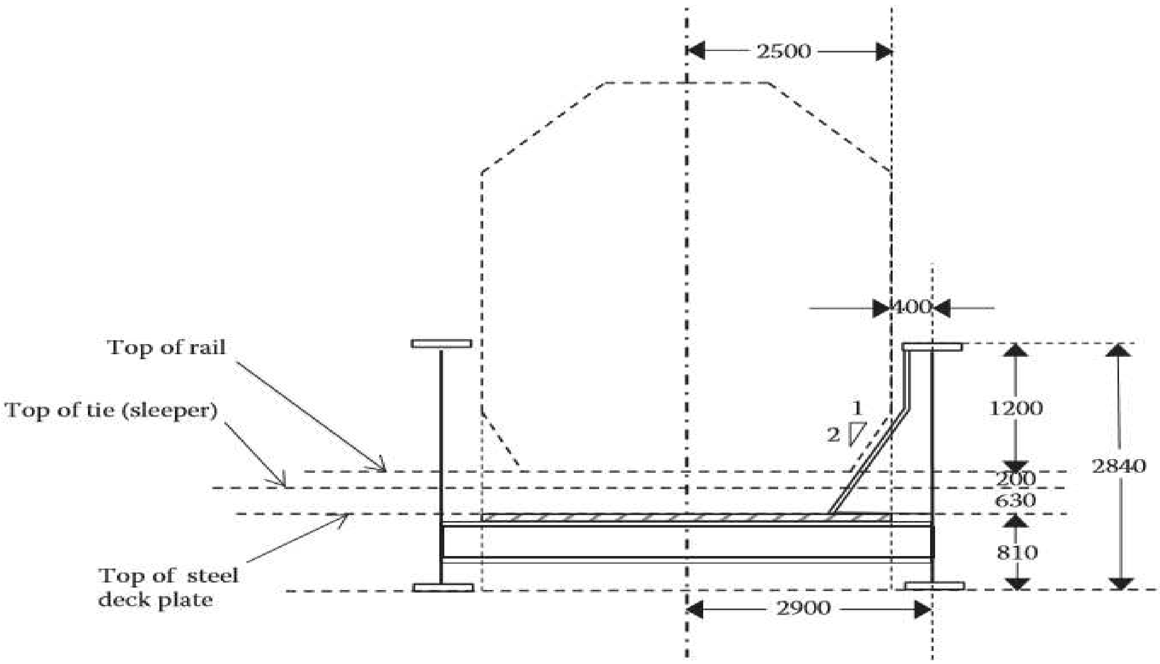
- Cooper's EM360 with alternate live load (445 kN axle load)
- Steel with F_y = 350 MPa

LAYOUT OF THE PROPOSED SUPERSTRUCTURE

Length of span between centers of bearings = 32,000 mm (assuming a 600 mm bearing plate).







Solution:

A. DECK PLATE

Step-1: Loads and Forces

Dead Loads

Track = (300(9.81)/(2.5 + 2(0.45)))/(1000) = 0.87 kPa (distributed to a 3.4 m deck plate width assuming 2.5 m tie with 1:1 distribution through ballast with db = 450 mm).

Ballast = $(1920)(9.81)(d_b)/(1000)^2 = 0.0188(d_b)$ *kPa.*

Assume a 600 mm ballast depth to account for up to 150 mm of future track raises.

Ballast = 11.3 KPa; use 12.0 kPa to account for contingencies related to ballast degradation and water retention.

Waterproofing = 50 (9.81)/1000 = 0.50 kPa.

Deck plate (assume a 24 mm plate) = $(24)7850(9.81)/1000^2$ = 1.85 kPa

w_{DL} *= 0.87 + 12.0 + 0.50 + 1.85 = 15.2 kPa.*

Model as continuous beam:

$V_{DL} = \frac{5}{4}\left(w_{DL} * beam\,spacing\right)$ = *1.25(15.2)(0.80) = 15.2 kN/m (maximum from two span continuous at interior support)*

$M_{DL} = \frac{w_{DL} * beam\,spacing^{2}}{8}$ = *0.125(15.2)(0.80)(0.80) = 1.22 kNm/m (maximum from two span continuous at interior support).*

Live Load

The longitudinal deck distribution width

= 915 + (450 + 24 − 180 – 12) = 1197 mm < 1525

Lateral distribution width

= 2500 + (450 + 24 − 180 – 12) = 2782 mm < 4250 mm

w_{LL} *= 445/((1.197)(2.782)) = 133.6 kPa (alternate live load axle)*

$V_{LL} = \frac{5}{4}\left(w_{LL} * beam\,spacing\right)$ = *1.25(133.6)(0.8) = 133.6 kN/m (maximum from two span continuous at interior support)*

$M_{LL} = \frac{w_{LL} * beam\,spacing^{2}}{8}$ = *0.125(133.6)(0.8)2 = 10.69 kNm/m (maximum from two span continuous at interior support).*

Vertical impact	
I_{max} *= 0.90(40.0%) = 36.0% (Figure)*	$V_{LL+I(max)}$ *= (1.36) 133.6 = 181.7 kN/m*
I_{mean} *= 0.65 (36.0) = 23.4% (Table)*	$M_{LL+I(max)}$ *= (1.36) 10.69 = 14.5 kNm/m*
	$M_{LL+I(mean)}$ *= (1.23) 10.69 = 13.15 kNm/m.*

Wind Forces

Wind load on train = 4.38 kN/m at 2.4 m above the top of rail (including lower 1.2 m of train not exposed to wind due to girder height, but this height will typically be only about 20% of train height)

w_{WLL} = *[4.38 (2.4)/1.5]/(0.90) = 7.7 kPa (wheel spacing = 1.5 m rail spacing distributed over 2(0.45) = 0.90 m width at deck plate elevation)*

$V_{WLL} = \frac{5}{8}\left(w_{WLL} * beam\,spacing\right)$ *=1.25(7.7 (0.8)/2) = 3.9 kN/m (maximum from two span continuous at interior support)*

$M_{WLL} = \frac{w_{WLL} * beam\,spacing^{2}}{8}$ *=7.7 (0.8)2/8 = 0.62 kNm/m (maximum from two span continuous at interior support).*

Load Combinations for Deck Plate Design (Table)

Load combination D1-A: DL + LL + I at 100% allowable stress:

$V_{max} = 15.2 + 181.7 = 196.9\ kN/m$

$M_{max} = 1.22 + 14.5 = 15.7\ kNm/m.$

Load combination D1-B: LL + I at allowable fatigue stress:

$M_{range} = 13.15\ kNm/m$

Load combination D2-A: DL + LL + I + WL at 125% allowable stress:

$V_{max} = 15.2 + 181.7 + 3.9 = 200.8\ kN/m$

$M_{max} = 1.22 + 14.5 + 0.62 = 16.3\ kNm/m.$

Due to the 2% increase in shear and 4% increase in bending moment from load case D1-A, load combination D2-A with F_L = 1.25 will not be considered.

Step-2: Deck Plate Design

Strength consideration;

$$T \leq 0.55F_yA_g$$

$$A_g \geq 15.7(1000)/(0.55(350)) = 81.6mm^2/mm$$

$$t_{dp} \geq \sqrt{6(81.6)} \geq 22.1mm\ for\ bending$$

Use a 22 mm plate with 3 mm allowance for corrosion = 25 mm.

Weight of deck plate = 25(5.0)(32.8)(7850)/1000 = 32,185 kg.

Fatigue consideration

$$T_{fat} = Sr_{fat} * A_{efat}$$

$$A_{efat} = 13.15(1000)/(165) = 79.7 \leq 81.6 \quad \textbf{ok!}$$

B. INTERMEDIATE FLOOR BEAMS

Step-1: Loads and Forces

Dead Loads

Track = 300(0.8)/2.5 = 96 kg/m distributed over 3.4 m (assuming 2.5 m tie and 1:1 distribution through ballast with d_b = 450 mm)

Ballast = (1920)(d_b)(800)/1000 = 1536(d_b) kg/m distributed over deck plate width ≃ 2[2900 − ((900 + 200 + 630)/2 + (810/2))] ≥ 2(2900 − 1270) ≥ 3260 mm ≥ 3.260 m (figure)

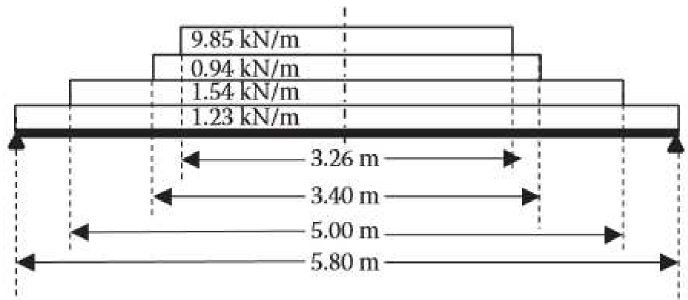
- Assume a 600 mm ballast depth to account for up to 150 mm of future track raises.

Ballast = 922 kg/m; use 1000 kg/m over 3.26 m length to account for contingencies related to ballast degradation and water retention

Waterproofing = 50(0.8) = 4 kg/m over 3.26 m length Deck plate (25 mm plate) = 25(800)7850/10002 = 157 kg/m over 5.0 m length

Floor beam = 125 kg/m (assumed weight of the floor beam section) over 5.8 m length.

The intermediate floor beam dead loading is shown below



V_{DL} = [9.85(3.26) + 0.94(3.4) + 1.54(5.0) + 1.23(5.8)]/2 = (32.11 + 3.20 + 7.70 + 7.13)/2 = 25.1 kN

M_{DL} = 9.85(3.26)(5.8/4 − 3.26/8) + 0.94(3.40)(5.8/4 − 3.40/8) + 1.54(5.00)(5.8/4 − 5.00/8) + 1.23(5.80)(5.8/4 − 5.80/8) = 33.48 + 3.28 + 6.35 + 5.17 = 48.3 kNm.

Live Load

Longitudinal distribution:

Floor beams spaced at 800 mm < axle spacing = 1500 mm

$$P = 1.15(445)\frac{(800)}{1500} = 272.9\,KN$$

Lateral distribution: No lateral distribution, P/2 applied at location of wheel loads

$$V_{LL} = 272.9/2 = 136.5$$

$$M_{LL} = 36.5(5.8-1.5)/2 = 293.5\,KNm$$

Vertical impact:

$I_{max} = (0.90)39.3\% = 35.4\%$ *(Figure)*

Imean $= 0.35(35.4) = 12.4\%$ *(Table).*

Rocking effect:	**Total impact:**
$RA = 0.20\ W\ (1525) = 305\ W$	$I_{max} = 35.4\% + 5.3\% = 40.7\%$
$RR = FR\ (5800)$	$I_{mean} = 12.4\%$
$FR = 0.053(W)$	$V_{LL+I(max)} = (1.407)\ 136.5 = 192.1\ kN$
$RE = FR\ (100)/W = 5.3\%.$	$M_{LL+I(max)} = (1.407)\ 293.5 = 413.0\ kNm$
	$M_{LL+I(mean)} = (1.124)\ 293.5 = 329.9\ kNm.$

Wind Forces on Loaded Superstructure (4.38 kN/m at 2.4 m above the top of rail)

$w_{WLL} = 4.38\ (2.4)/1.5 = 7.0\ kN/m$

$V_{wLL} = (7.0\ (5.8)/2) = 20.3\ kN$

$M_{wLL} = 7.0\ (5.8)2/8 = 29.4\ kNm.$

Load Combinations for Intermediate Floor Beam Design

Load combination D1-A: DL + LL + I at 100% allowable stress:

$V_{max} = 25.1 + 192.1 = 217.2\ kN = R_{max}$

$M_{max} = 48.3 + 413.0 = 461.3\ kNm.$

Load combination D1-B): LL + I at allowable fatigue stress:

$M_{range} = 329.9\ kNm.$

Load combination D2-A: DL + LL + I + WL at 125% allowable stress:

$V_{max} = 25.1 + 192.1 + 20.3 = 237.5\ kN = R_{max}$

$M_{max} = 48.3 + 413.0 + 29.4 = 490.7\ kNm.$

Due to the 9% increase in shear and 6% increase in bending moment, load combination D2-A with FL = 1.25 will not be considered.

Step-2: Intermediate Floor Beam Design

$A_{web} \geq 217.2(1000)/(0.35(350)) \geq 1773\ mm^2$

$S_{min} \geq 461.3(1000)^2/(0.55(350)) = 2396 \times 10^3\ mm^3$

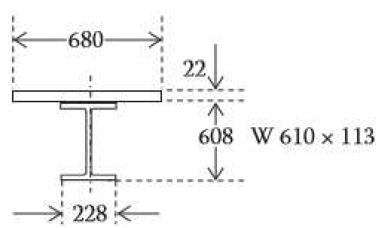
$S_{mean} \geq 329.9(1000)^2/(165) = 1999 \times 10^3\ mm^3.$

Effective width for a 25 mm deck plate

$$b_{eff} \leq (2_{tp})(0.43)\sqrt{\frac{E}{F_y}} = (2)(25)(0.43)\sqrt{\frac{200{,}000}{300}} = 514mm$$

$$b = b_{eff} + b_{beam} = 514 + b_{beam} \leq \textit{floor beam spacing} \leq 800mm$$

Try W 610 × 113:



Intermediate Floorbeam Section Properties (22 mm Deck)

Section	A (mm²)	y (mm)	A_y (mm³)	$y_s - y$ (mm)	$A\ (y_s - y)^2$ (mm⁴)	I_o (mm⁴)
W 610 × 113	14,400	304.0	4377.6×10^3	160.5	371.0×10^6	875×10^6
22 mm deck plate	14,960	619.0	9260.2×10^3	–154.5	357.1×10^6	603.4×10^3

$$b = 514 + 228 = 742 \leq 800mm................ok!$$

Maximum and mean (fatigue range) stresses:

$$y_s = \frac{(4377.6 + 9260.2)(1000)}{14400 + 14960} = 464.5mm$$

$I_g = (371.0 + 357.1 + 875 + 0.60)10^6 = 1603.7 \times 10^6\ mm^4$

The intermediate floor beams at the center of each bracing panel (in the plan view above) will be connected to the bottom lateral bracing at mid-span. Assuming 2–25 mm dia. holes:

$\boldsymbol{I_n} = 1603.7 \times 10^6 - 2(25)(17)((17/2)–464.5)^2 = (1603.7 - 176.8) \times 10^6 = 1427.0 \times 10^6\ mm^4$

$\boldsymbol{S_t} = 1603.7 \times 10^6/(608 + 22 - 464.5) = 9690.0 \times 10^3\ mm^3$

$\boldsymbol{S_b} = 1427.0 \times 10^6/(464.5) = 3072.1 \times 10^3\ mm^3$

$\boldsymbol{A_{web}} = (608 - 2(17))(11) = 6314\ mm$

$\boldsymbol{\sigma_{max}} = 461.3 \times 10^6/3072.1 \times 10^3 = 150.2\ MPa \le 0.55(350) \le 192.5\ MPa$, *OK.*

$\boldsymbol{\sigma_{range}} = 329.9 \times 10^6/3072.1 \times 10^3 = 107.4\ MPa < 165\ MPa$, *OK.*

$\boldsymbol{\tau_{max}} = 217.2\ (1000)/(6314) = 34.4\ MPa \le 0.35(350) \le 122.5\ MPa$, *OK.*

LL + I deflection:

$$P_{LL+I} = (1.407)1.15(445)(800)/1500 = 384.0\ kN$$

$$\Delta_{\max} = \frac{384(1000)(2150)(3(5800)^2 - 4(2150)^2}{24(200{,}000)(1603.7 * 10^6)} = 8.9mm$$

$$\Delta_{all} = \frac{span}{640} = 9.1mm.................ok!$$

Use W 610 × 113 with for intermediate floor beams.

Step-3: Intermediate Floor Beam Weight

Use W 610 × 113 intermediate floor beams. Weight of intermediate floor beams = 113(5.8) ((32,000/800) − 1) = 25,560 kg.

C. END FLOOR BEAMS

End floor beams are designed for dead load, live load, and wind load (on live load). End floor beams are also designed as jacking beams. The end floor beams as jacking beams will reverse stresses in the flanges and will typically govern design of the relatively small end floor beams lifting considerable dead load.

EXERCISE 5-1:

1. Compare railway Vs. highway bridge on the following issues
    a) Intensity of live loads
    b) Design impact load
    c) Interruption in service
    d) Design philosophy
2. Discuss importance the following materials for the current railroad bridge construction practices; and compare their degree of acceptance?
    a) RC structures
    b) PSC structures
    c) Steel structures
    d) Brick and masonry structures
3. Suppose you are part of railway bridge 'design team';
    a) How to select whether the bridge deck is open or ballasted?
    b) How to decide the position of carriageway?
4. Design reinforced concrete deck with the following information.

GENERAL INFORMATION

Here below is a simply supported ballasted deck plate girder bridge with 30m span. Railroad width is 9860 mm from curb to curb which carries double track. Providing 170mm thick ballast and a standard tie 2.45m long and 205mm wide spaced at 610mm. provide concrete deck thickness of 300mm.

LOADING AND MATERIALS

- Cooper's EM360 with alternate live load (445 kN axle load)
- concrete with *f'c= 30 MPa*
- Steel with F_y = 350 MPa

LAYOUT OF THE PROPOSED SUPERSTRUCTURE

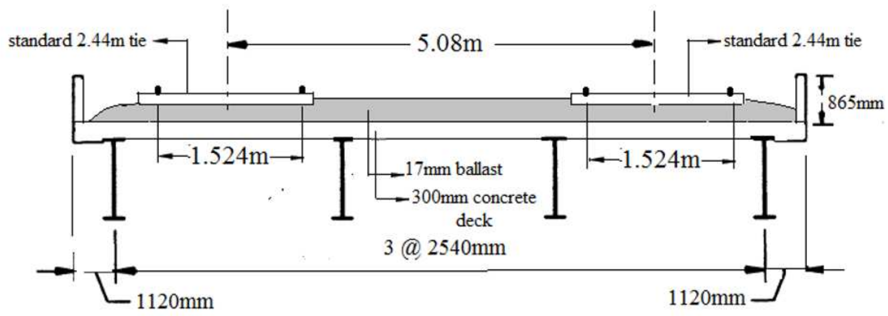


Figure 0-1: deck plate girder bridge cross-section

MODULE-SIX

6. RAILWAY SIGNALING

Introduction

In railway terminology signaling is a medium of communication between the station master or the controller sitting in a remote place, in the office, and the loco driver of a train; to control and regulate the movement of trains safely and efficiently.

By far the most dangerous movement of trains is when passing over points and crossings. It is vital that trains are kept from moving into any position of danger and are only allowed to precede when all train movements are properly coordinated and when the route ahead is properly set and clear.

Types of Signaling Systems in Railways:

a) Time interval method
- The spacing of signals depends to a large extent on the minimum time interval that is required between trains, known as the headway.
- On a main line railway where mixed stock passes at different speeds, spacing of signals becomes a complex subject.
- On rapid transit railways and light railway systems it becomes more straightforward as the trains tend to have the same movement characteristics.

b) Space interval method
- The specified lengths of line between boxes, stations or junctions, termed block sections, formed a space interval between trains.
- Not more than one train should be allowed on one line in a block section
- A principle that continues to be the basis of modern signaling today.

All modern signaling systems have the following basic objectives:

- To control trains in a safe manner for the conditions ahead.
- To maintain a safe distance to any train ahead or dead end ahead.
- To prevent the setting of conflicting movements.
- To ensure that points are locked in the correct position.
- To enable trains to operate to the headway required.
- To enable trains to operate to the scheduled speed with minimum disruption consistent with safety.

6.1. Basic Sub-Systems in Signaling

Railway signaling can be made fail-safe for all conditions providing the following:

- Track circuits.
- Point locking and detection.
- Interlocking.

6.1.1. Track Circuits

Track circuits on many railway networks are used to continuously detect the absence of a train within its designated block hence its primary failure mode is to indicate the presence of a train or a metallic obstacle crossing the rails within the block. Block occupancy is triggered by a 'shunting' shorting of the track circuit. A basic DC track circuit is defined at its extremities by insulated joints.

A basic track circuit has power applied to both rails at one end and has a relay coil across the rails at the other end. In the absence of a train the relay is energised by the current from the power source. An energised relay tells the signalling system that the track section is clear. When a train occupies the track within the block, the wheels and axles (shunt) short circuit the track so that most of the current is passed through the wheels, as this presents the path of least resistance, and not to the relay. In this situation there is not sufficient current to pick up the relay and the track is indicated as occupied.

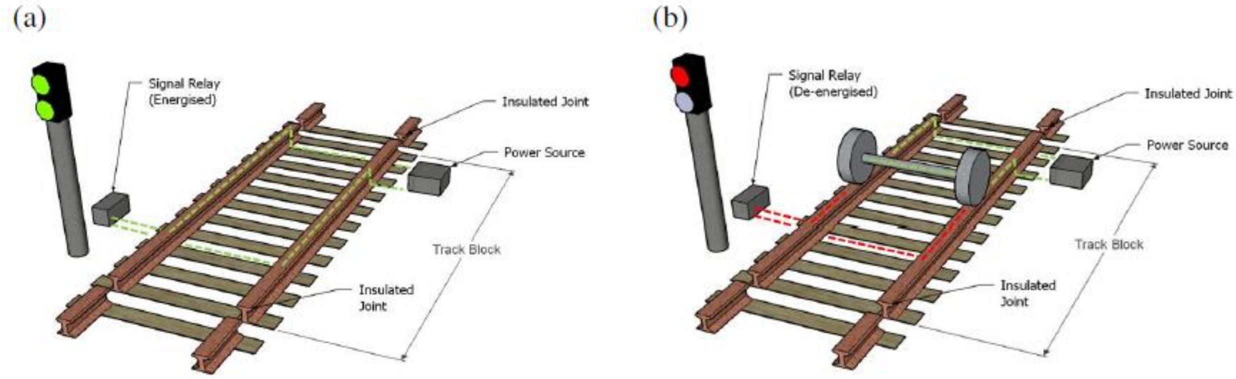


Figure 6-1: Basic track circuits (a) unoccupied and (b) occupied

Both circuits have a default fail safe which is to signal that the block is occupied. Any short circuit, caused by the presence of a train or a break in the track will lead to the relay failing and signalling that the block is occupied/unsafe to enter. Both circuits incorporate a good degree of failsafe; however, they both rely on there being good electrical conductivity across the wheel/rail interface.

6.1.2. Locking and Detection

The potential hazards to safe operation from points arise from the possibility that:

a. They will be set in the wrong direction or that
b. They will come open during the passage of a train.

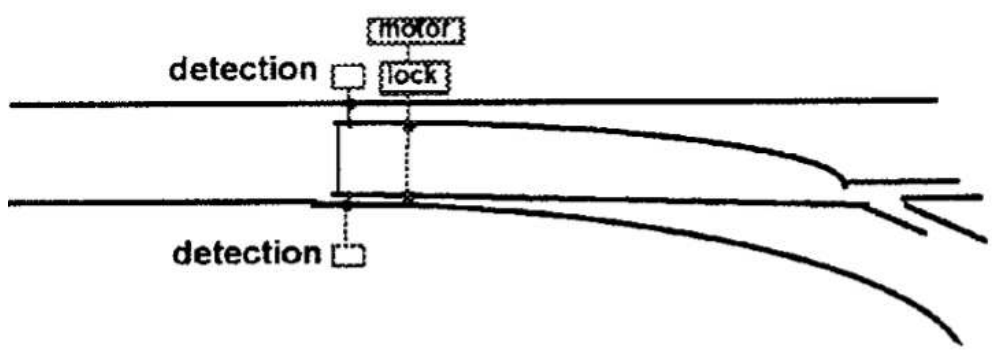


Figure 6-2: Diagram showing point locking and detection.

With modern point motor assemblies, devices are integrated into the mechanism which:

- locks the points in position once they are fully home
- Detects, electrically, that they are actually fully closed.
- Detects the presence of an obstruction, such as a discarded can, in the point blades.

6.1.3. Interlocking

In principle, interlocking is introduced to prevent signalmen accidentally clearing a signal before points are properly set or clearing signals that would allow a conflicting movement. In manual boxes a series of sliding bars is connected to the levers. These bars have notches and dogs in them which will only allow operation of the signals in correct relationship to other signals and when relevant points are correctly set.

6.2. Minimum Headways

The headway of a particular train service is the minimum time interval that can be run between trains. The most critical point in this consideration is at a station because the station stop or dwell time must be taken into consideration.

The practical minimum dwell time at any station will vary considerably and will depend upon:

- the number of passengers alighting and getting on,
- the number and spacing of doors on the rolling stock,
- width of platform,
- the number of waiting passengers obstructing movement,
- the proportion of passengers carrying luggage and the percentage of passengers who are familiar daily commuters.

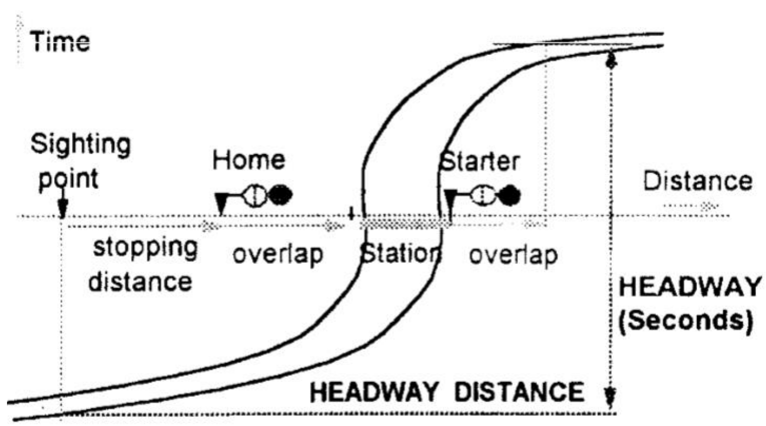


Figure 6-3: Headway distance and time.

Note: Headway is measured from the time at which the driver reaches the sighting point of the first signal to a point at which the tail of the train clears the overlap of the signal ahead.

6.3. Types of Signals

6.3.1. Home and Distant Signals

To give early warning of signals at danger therefore, all signals were classed as either *'home'* or *'distant'*. On running main lines all stop or home signals were provided with a distant signal in advance which indicated whether or not the following home signal was clear.

- When using semaphore signals the arms of the distant signals were usually painted yellow with a black Vee and notched at the end.
- 'Stop' or 'Home' signals usually had red painted arms with a white band and square cut ends, their lenses being always red and green only.

Approaching junctions, twin posted home and distant signals were provided which were thus able to warn the driver which path was clear.

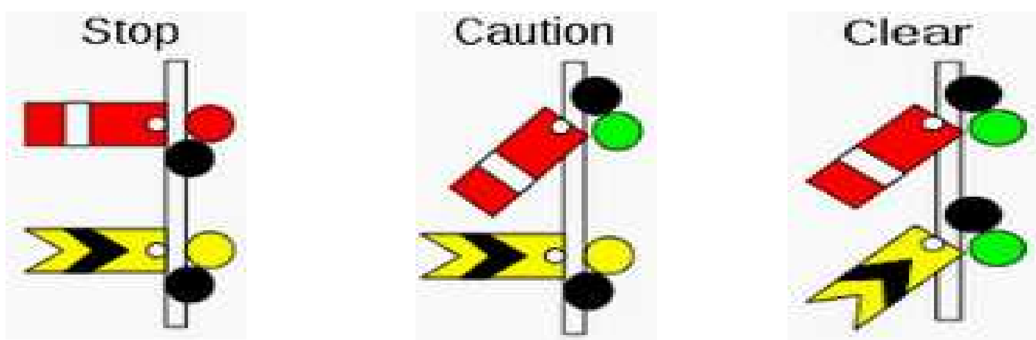


Figure 6-4: Semaphore and Warner signals

6.3.2. Subsidiary Signals

Subsidiary signals are used for controlling low speed shunting and other non-running movements both on main lines and in yards. These signals are usually small and used:

- For coupling up locomotives to trains and working in the reverse direction to normal.
- Where signals are difficult to see because of obstructions or tight curves,

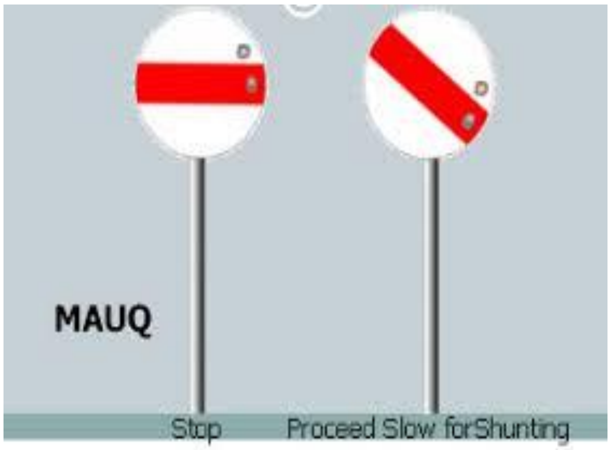


Figure 6-5: shunting signals

6.3.3. Colour Light Signaling

Semaphore signaling began to be replaced by electrically powered colour lights when DC electrification began to be installed. The initial cost of installing colour light signaling is high but the advantages are considerable. Pulling signal and point levers by hand was a skillful but heavy task and severely limited the distances that signal boxes could be apart. Once points and signals were operated electrically, the number of signal boxes could be reduced dramatically and much longer lengths of railway could be controlled from one location.

On the basis of indications lumps, colour light signals can be classified in to:

- Two aspect colour light signaling
- Three aspect colour light signaling
- Four aspect colour light signaling

6.3.3.1. Two Aspect Colour Light Signaling

The simplest system of electric signaling is to have signals with two lamps each with a fixed coloured lens,

- red or green for a stop or home signal, and
- Orange or green for a repeater or distant signal.

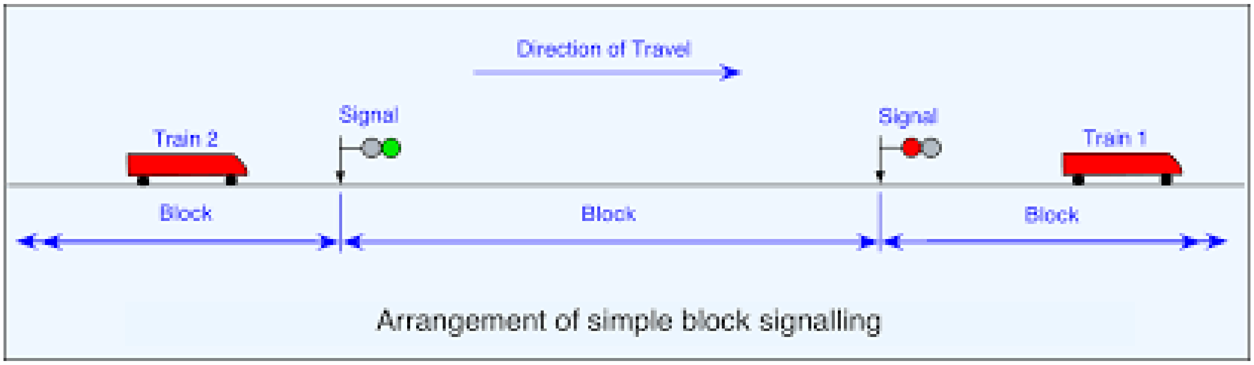


Figure 6-6: Two aspect colour light signaling

On rapid transit, metro and light rail systems, two aspect signaling is usually adopted because the braking distances are short due to low speeds and high braking rates, hence the distance needed for the driver to stop the train is usually such as to be able to see the signal in time.

6.3.3.2. Three Aspect Colour Light Signaling

Where train speeds are higher, or headways are closer three aspect signaling can be used.

- This allows trains to approach closer and effectively combines the distant signal of the signal ahead with the running signal.
- These signals show red, yellow or green indications,

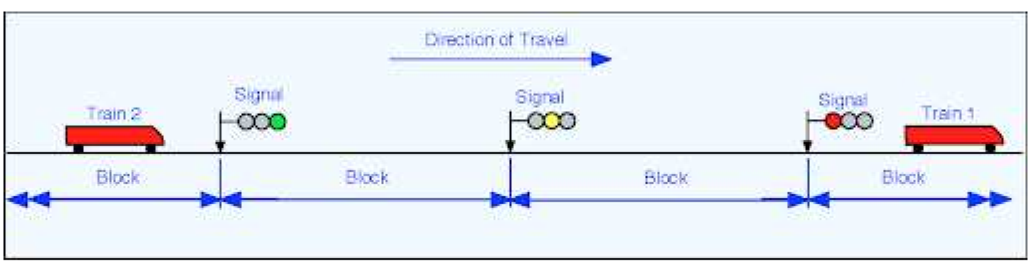


Figure 6-7: Three aspect colour light signaling

6.3.3.3. Four Aspect Colour Light Signaling

The four aspect system allows trains at higher speeds to slow down earlier and hence to get closer to the train ahead in a controlled manner. Reading from the top down, the four aspects are yellow, green, yellow and red.

- Having red at the bottom is that it can be placed at driver's eye level and there is no lens hood below it which might assist snow to build up and obscure the light.
- It is a 'fail-safe' principle that maintenance of a red light must take precedence over everything else.

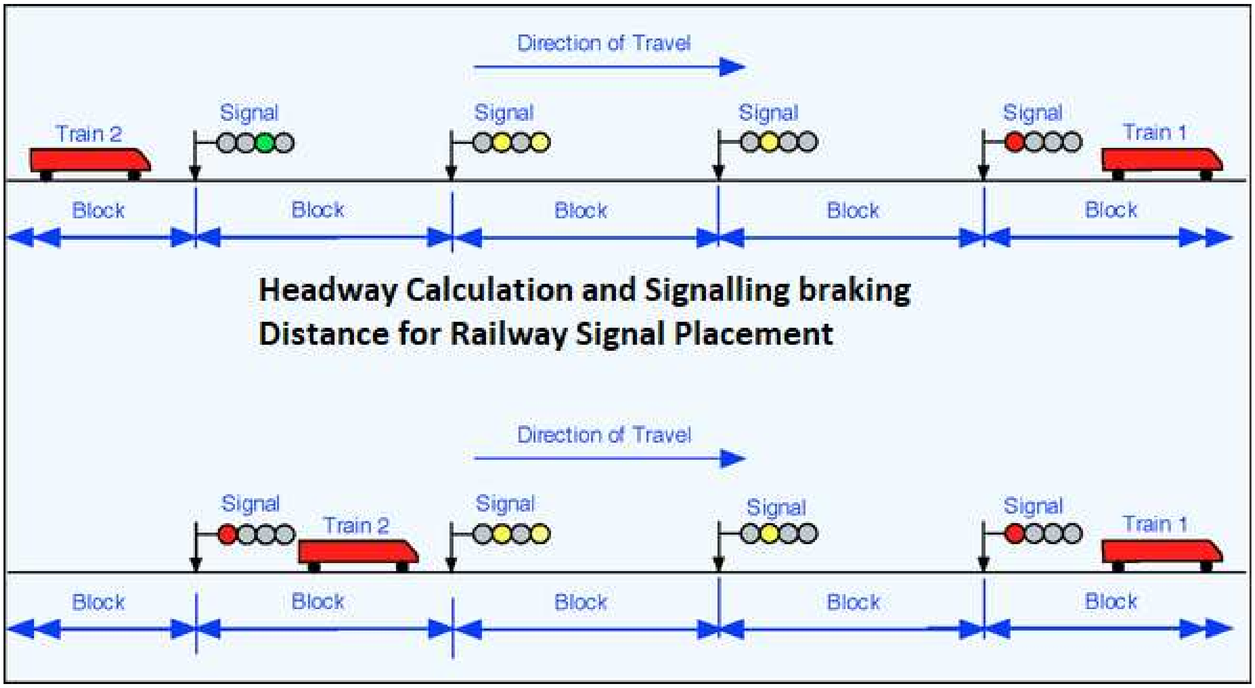


Figure 6-8: Four aspect signaling.

The sequence of four aspect signaling is as follows:

- Green — Continue at full speed.
- Double yellow — Proceed at caution, reducing power.
- Single Yellow — Power off , controlled braking, ready to stop.
- Red — Stop.

6.3.4. Transmission Based Signaling

Metro and light rail systems have introduced automatic running of trains between stations that are compatible with the computer controlled signaling and safety systems.

These combining systems are often known as

- Automatic Train Operation (ATO),
- Automatic Train Scheduling (ATS), and
- Automatic Train Protection (ATP).

Interlocking of points, signals and routes is still a very important requirement to ensure that trains only operate when it is safe to do so.

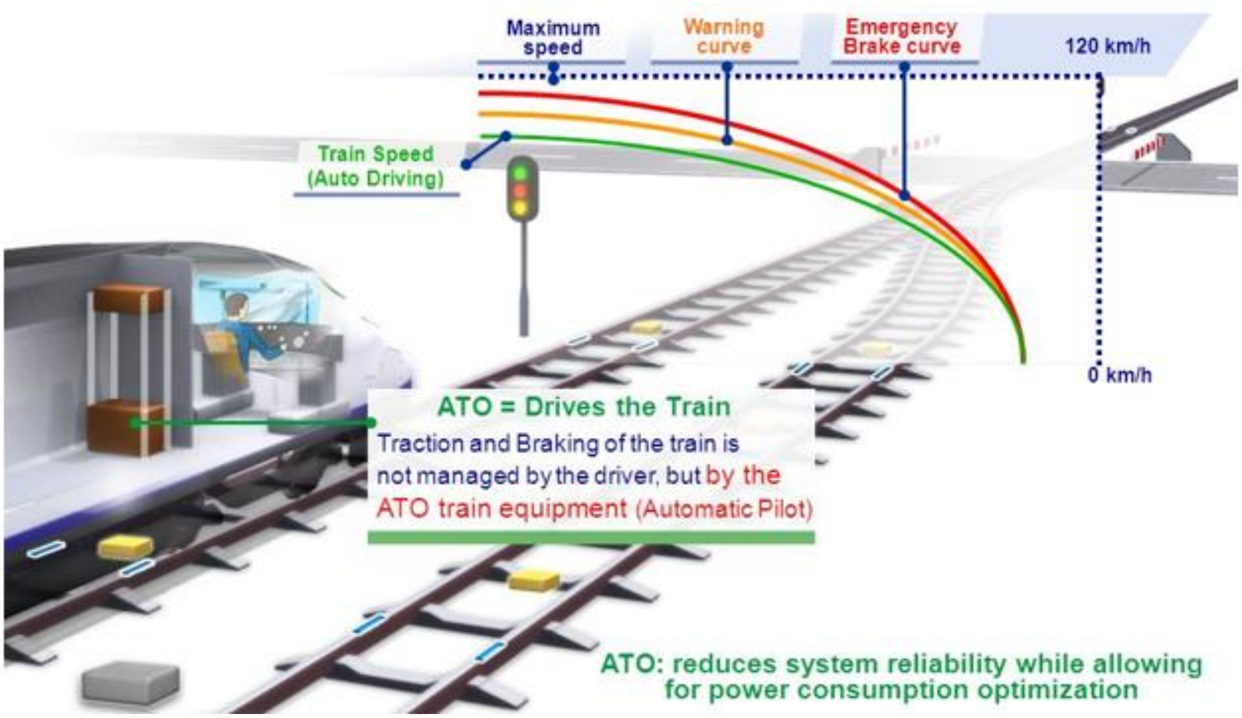


Figure 6-9: Automatic Train operation (AREMA, 2015)

6.4. Safety Standards

The component parts of any signaling and control system must each meet the required safety level, depending on the likely results of failure. For instance;

- The level of safety required for a component, the failure of which could lead to death, must be of a higher standard than for a component or system the failure of which could cause people to be misled into a dangerous situation.

Proof of Safety and compliance with safety standards:

- Needs to be applied to all signal and control equipment, hardware and software, system design, testing, commissioning, maintenance and replacement.
- Consideration needs to be given to the possible action that can be taken when incidents downs occur.

EXERCISE 6-1:

1. By taking the case of Addis LRT; discuss consistency of the signal, communication, and control system with international 'urban-signaling' practices?
2. What are the objectives of interlocking? Explain the tappet and lock system of interlocking.
3. Compare the types of controlling the movement of trains, time interval & space interval, for single and double lines?
4. Discuss on advancements in ease of train control; through two aspect, three aspect, and four aspect signaling?
5. Which specific problems are addressed by the introduction of transmission based signaling in to the railway industry?

MODULE-SEVEN

7. SWITCHES AND CROSSINGS

Introduction

In railway engineering, direction is changed by switching devices, or switches, defined as the equipment or a devices in the railway superstructure that allow a rail vehicle to change from one track to another without interrupting its course.

This applies to all railways from the most complicated reversible layouts at terminal stations to simple single track tramway switch passing loops. Any assembly of points and crossings is called a layout. Some layouts occur frequently and have acquired their own names. In general, the longer the switch point rail, the more gradual the angle of divergence from the main track and the faster the rail vehicle can travel through it.

It is estimated that 20 to 40 percent of the track maintenance budget is spent on the inspection, maintenance and renewal of switch and crossing. In comparison, the maintenance cost of one switch or crossing equals the maintenance costs of 300 to 500 meters of plain track, This relative high expenditure is mainly due to the nature of switch and crossing that makes them absolute and relative more expensive to maintain than plain track.

7.1. Turnout

Turnout is the simplest combination of switches and crossings which have movable component and flange way gap due to this reason we call it special track structure from railway track. .

- The points or switches aid in diverting the vehicles and the crossings provide gaps in the rails so as to help the flanged wheels to roll over them.
- A train moving from the switch to the frog (i.e. it can be directed to one of the two paths depending on the position of the points) is said to be facing-point movement and
- A train moving from the crossing to the switch is said to be trailing-movement.

From many types of turnout arrangements, straight turnouts with rigid frogs and split switches are the most common.

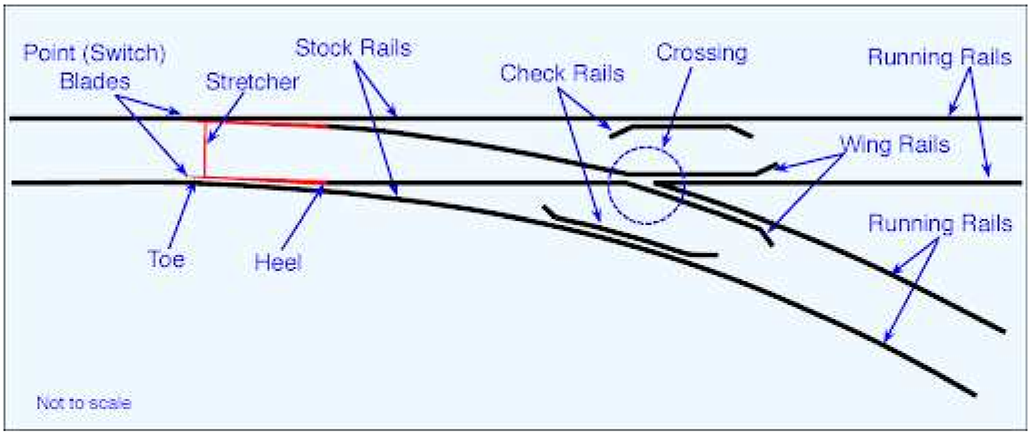
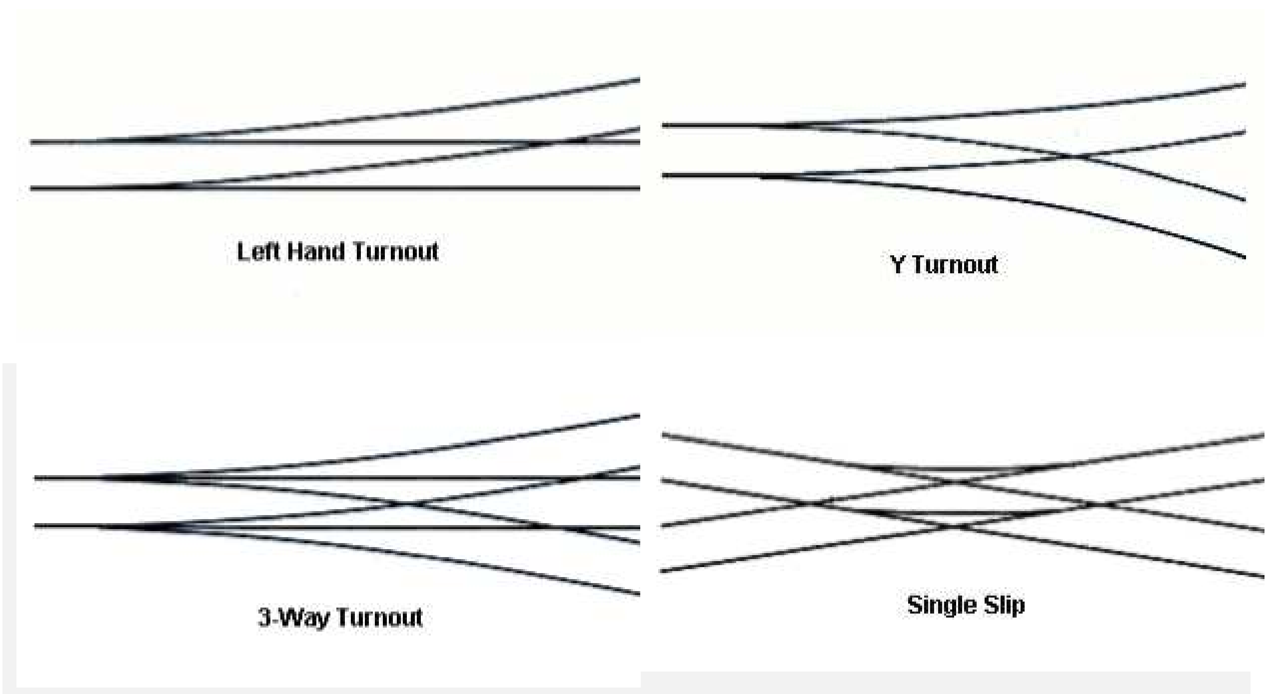
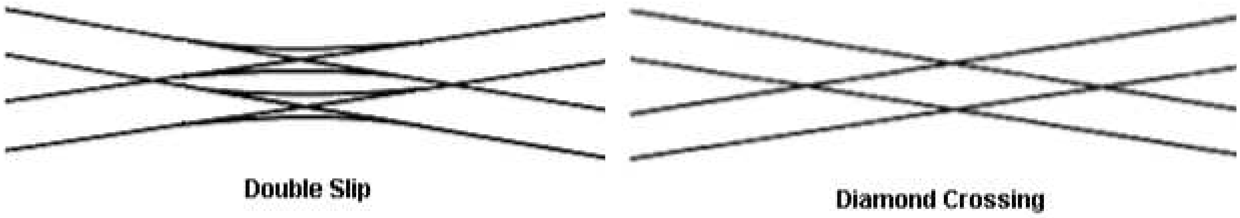


Figure 7-1: simple turnout

7.1.1. Types of Turnouts

There are a number of standard layouts or types of turnouts including the following:





7.2. Crossing

A crossing or frog is a device introduced at the point where two gauge faces cross each other to permit the flanges of a railway vehicle to pass from one track to another.

To do such task; gap is provided from throw to the nose of crossing and the Check rails control the alignment of the wheel-set so that it is not possible for the wheel moving across the gap in the throat of the crossing to strike the nose of the crossing or to take a wrong path. This makes the vehicle negotiate the crossing smoothly.

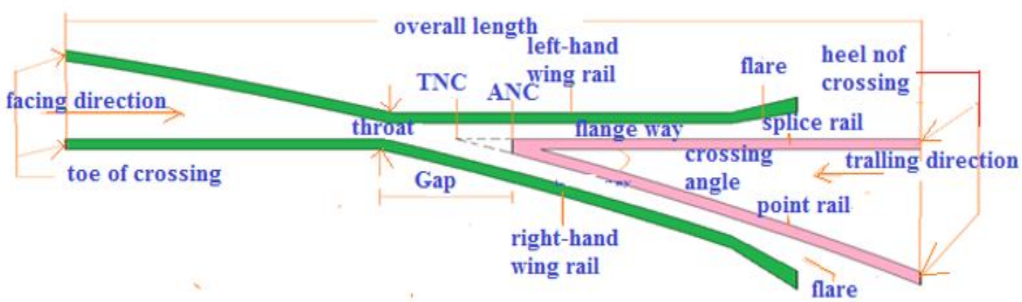


Figure 7-2: Details of crossing

7.2.1. Types of Crossings

Crossing types on the basis of angles formed when tracks cross each other:

a. An acute angle crossing or 'V' crossing
b. An obtuse or diamond crossing
c. A square crossing (rarely used in actual practice)

Crossing types on the basis of purpose for which it is manufactured:

a. **Built-up crossing:** Two wing rails and a **V** section consisting of splice and point rails are assembled together by means of bolts and distance blocks to form a crossing. Their initial cost is low and that repairs can be carried out simply by welding or replacing each constituent separately. They are subject to wear however, particularly at the tip of the point rail and where the point and splice rail bear against one another.
b. **Part-welded crossing:** Consists essentially of the same four rails as a built-up crossing and is usually made of standard rail. It's strong enough to take thermal loads and it can be welded into CWR.As a disadvantage; when one element fails under traffic and has to be cut out and re-welded rather than re-bolted into position.
c. **Cast steel crossing:** This is a one-piece crossing with no bolts and, therefore, requiring very little maintenance. Comparatively, it is a more rigid crossing since it consists of one complete mass. The initial cost of such a crossing is, however, quite high and its repair and maintenance pose a number of problems
d. **Austenitic Manganese Steel (or AMS) crossing:** This type of crossing is favored by many railways due to its very high wear resistance and long life. Also due to being monolithic there is no relative movement of components and the ride is generally very good.

AMS crossings do have some disadvantages.

- Casting as a process is always subject to internal cracking due to cooling
- When faults do arise in service, the castings are much heavier and difficult to handle during a limited possession than built-up crossings.

7.3. Switches

It is an arrangement of special structure which consists of the pair of linked tapering rails, known as points (switch rails or point blades), lying between the diverging outer rails.

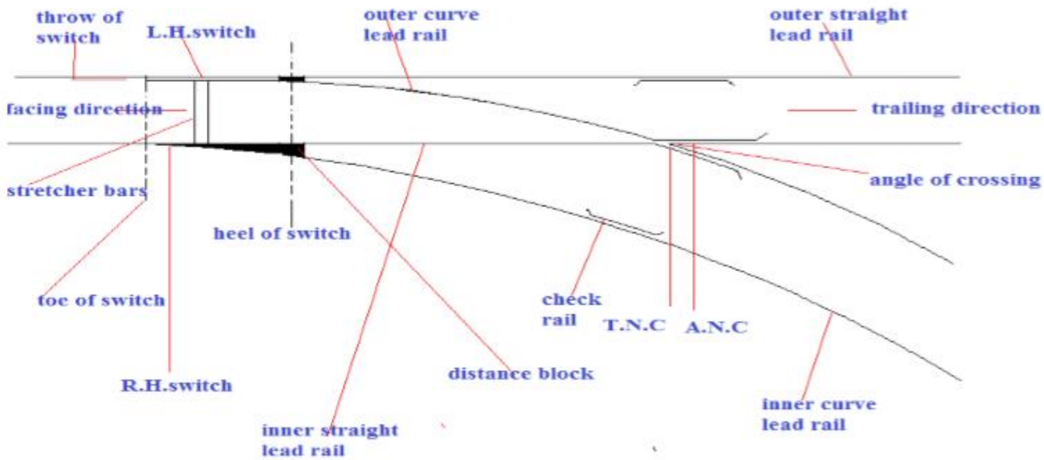


Figure 7-3: split switch

A mechanism is provided to move the points from one position to the other:

- Historically, this would require a lever to be moved by a human operator, and some switches are still controlled on this way.
- However, most are now operated by a remotely controlled electric motor or by pneumatic or hydraulic mechanism.
- Tongue rails are tapered on most switches, but on stub switches they have square ends and machined to a very thin section to obtain a snug fit with the stock rail.

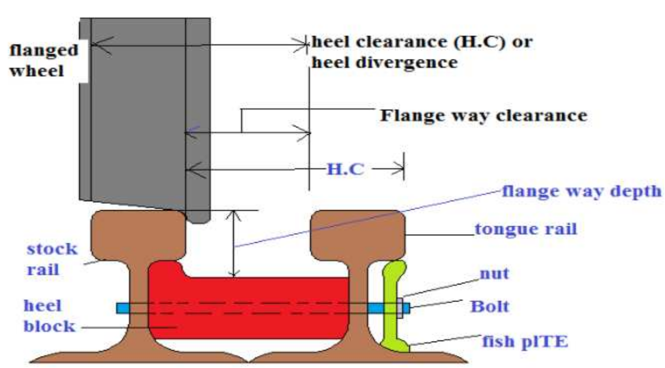


Figure 7-4: section at heel of tongue rail

7.3.1. Switches – Components

A set of points or switches consists of the following main Components:

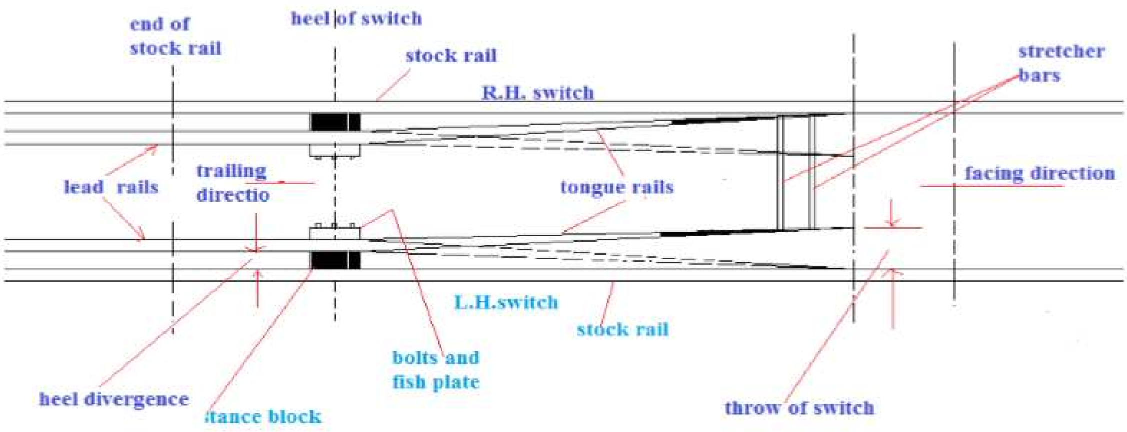


Figure 7-5: fixed heel type switch

I. ***Stock rail:-*** are the main rails of the track to which the tongue rails are fit closely

II. ***Stretcher bar: -*** connect the toe of the tongue rails so that both the tongues move through the same distance or gap.

III. ***Heel blocks or distance blocks: -*** connects the heel of the tongue rail with the stock rail,

IV. ***Sliding plate :-*** are the special plates which are provided for supporting and sliding the tongue rails (to move toward and away from stock rails)

V. ***A gauge tie plate: -*** fix gauges and ensure correct gauge at the points.

VI. ***Switch motor: -*** is an electric, hydraulic or pneumatic mechanism that aligns the points with one of the possible routes.

- It also includes electrical contacts to detect that the switch has completely set and locked.
- If the switch fails to do this, the governing signal is kept at red (stop).

VII. ***Points lever: -*** or ground throw (switch stand) is a lever that are used to align the points of a switch manually.

VIII. ***Point indicators: -*** to see the line of a switch from a distance, especially at night.

7.3.2. Types of Switches

7.3.2.1. Stub Switch

No separate tongue rail is provided and same portion of the track is moved from one side to the other side. First developed for steam railways and its straight and diverging tracks were completely separate and side by side. Stub switches are no more in use on current railway system.

7.3.2.2. Split Switch

A split switch essentially consist of two parts a stock rail and a tongue rail.

Classification of split switch on the basis of fixation at heel:

1. **Loose heel type or articulated type**

The switch or tongue rail ends at the heel of the switch to enable movement of the free end of the tongue rail: the tongue rails are joined to lead rails by means of fish plates; this type of switch is suitable for the short length of switches; the use of these switches is not preferred as the discontinuity of the track at the heel causes weakness in the structure

2. **Fixed heel type or spring type or flexible type**

The tongue rail does not end at the heel of the switch, but extends further and is rigidly connected. The movement at the toe of the switch is made possible on account of the flexibility of the tongue rail.

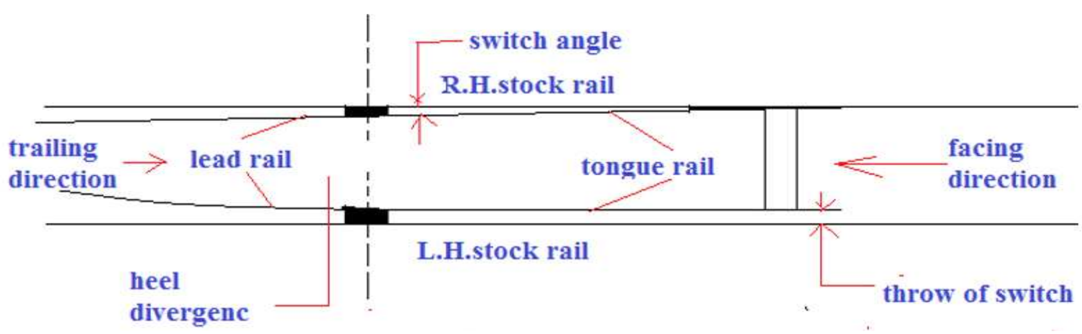


Figure 7-6: line diagram of fixed heel type switch

Classification of split switch on the basis of shape of switch rail:

a. under Cut Switches

In case the height of the stock and tongue rail is same, it is desirable to cut out a portion of flange at the foot of the stock rail; so that toe of tongue rail is accommodated under head of the stock rail. This type of switch becomes weak because flange portion is cut out.

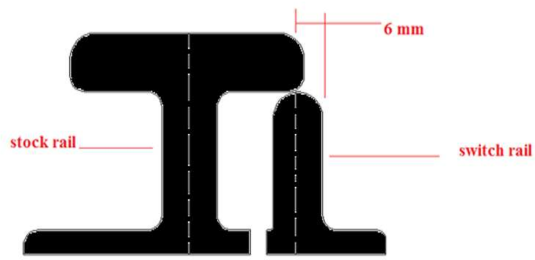


Figure 7-7: undercut switch

b. overriding Switches

The stock rail occupies the full section and the switch rail is kept higher than the stock rail from the heel to the point towards the toe where the planning starts. This is done to eliminate the possibility of splitting caused by any false flange moving in the trailing direction.

Override switch design is considered to be an economical and superior design since the stock rail is uncut, it is much stronger. Manufacturing work is confined only to the tongue rail, which is very economical. Although the tongue rail has a thin edge, it is supported by the stock rail for the entire weakened portion of its length.

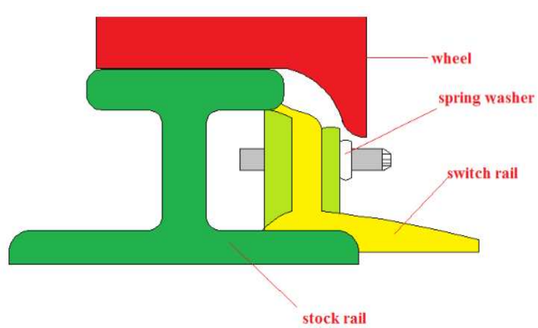


Figure 7-8: overriding switch

c. straight cut Switches

In this type the tongue rail is cut straight in the line with the stock rail. This is done to increase the thickness of toe of the tongue rail, which as result increases its strength. This type of switch is suitable for Bull Headed (BH) rails.

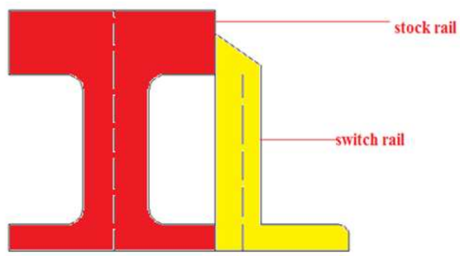


Figure 7-9: straight cut switches

7.4. Geometric Design Methods of Turnout

There are two standard methods prevalent for designing a turnout: Coles method and IRS method. The important terms used in describing the design of turnouts are defined as follows;

a) ***Curve lead (CL):-***
   - The distance from the tangent point (T) to the theoretical nose of crossing (TNC)
   - Measured along the length of the main track.

b) ***Switch lead (SL):-***
   - the distance from the tangent point (T) to the heel of the switch (H.S)
   - measured along the length of the main track

c) ***Lead of crossing (L)***
   - Distance between T.N.C and the heel of switch (H.S) measured along the length of the main track.
   - Lead of crossing (L) = curve lead (CL) – switch lead (SL)

d) ***Angle of crossing (α):-***
   - The angle between the main line and the tangent of the turnout line.

e) ***Angle of switch (β):-*** the angle between the switch rail

f) ***Gauge (G):-***the gauge of the track.

g) Number of crossing(N):

h) ***Radius of outer curve (R_o)***
i) ***Radius of center line of turnout (R)***,
j) ***heel divergence or clearance (d)***

7.4.1. Cole's Method -1

All the three lead CL, SL, and L are calculated in this method and crossing angle is calculated by using right angle method, in this simple design of turnout, a crossing curve is considered to start from an imaginary tangent point head actual toe of the switch and end at theoretical nose of crossing. These arrangements result in the formation of three kinks, namely

- a kink at the toe of a switch , this is because the switch rail is straight
- a kink at the heel of switch , this is because the switch rail is not tangential to the curve
- a kink at the toe of crossing, this because the curve is carried theoretically up to T.N.C. but the crossing actually is straight

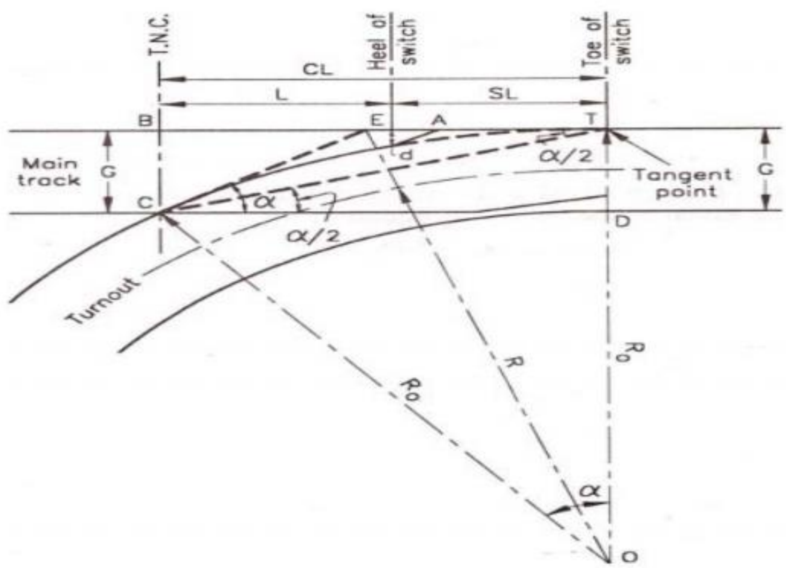


Figure 7-10: Cole's method -1 turnout design

1) curve lead

From right angle (Cole's method) Number of the crossing (N) = cot α from triangle TBC (TCD)

Tan (α/2) =G/CL

CL=G cot (α/2)

From triangle CDO

$CL^2 + (Ro- G)^2 = Ro^2$

$CL^2 = G (2Ro- G)$

$CL^2 = 2RoG$….neglecting G^2 which is small as compared to 2GRo

$CL^2 = 2RoG$

From line BT

CL= BA + AT

CL= BA + AC : - AC=AT

CL= G cot α +G cosec α

$$CL = G\sqrt{1+\cot^2\alpha} + \cot\alpha$$

$$CL = G\sqrt{1+N^2} + GN ...where..N = \cot\alpha$$

Then approximately CL becomes;

$$CL = 2GN \tag{7-1}$$

2) R- radius

From triangle OCD

Ro=CL/sin α

Ro=TD+OD

Ro= G + CL cot α…………..where CL=2GN and N= cot α

Ro= G + 2GN. N… because CL is actually slightly greater than 2GN and hence more accurate v alue of Ro is given by

Ro=1.5 G + 2GN^2

7-2

$$R = R_o - \frac{G}{2}$$

3) **Switch lead (S.L)**

$SL^2 + (Ro- d)^2 = Ro^2$

$SL^2 = d(2Ro- d)$

$SL^2 = 2Rod$ ……….as d^2 is very small as compared to2Rod

7-3

$$SL = \sqrt{2R_o * d}$$

4) **Lead or crossing lead (L)**

L=CL-SL

$$L = G\cot(\alpha/2) - \sqrt{2R_o * d}$$

7-4

$$L = 2GN - \sqrt{2R_o * d}$$

5) **Heel divergence (d)**
From equation (7-3)

7-5

$$d = \frac{SL^2}{2R_o}$$

7.4.2. Cole's Method -2

In this method only the crossing lead (L) is calculated and the curve is tangent to the switch heel rail. It springs from the heel of switch and ends at T.N.C with the help of this method, out of three types of kinks; a kink which is formed at the heel of the switch is removed.

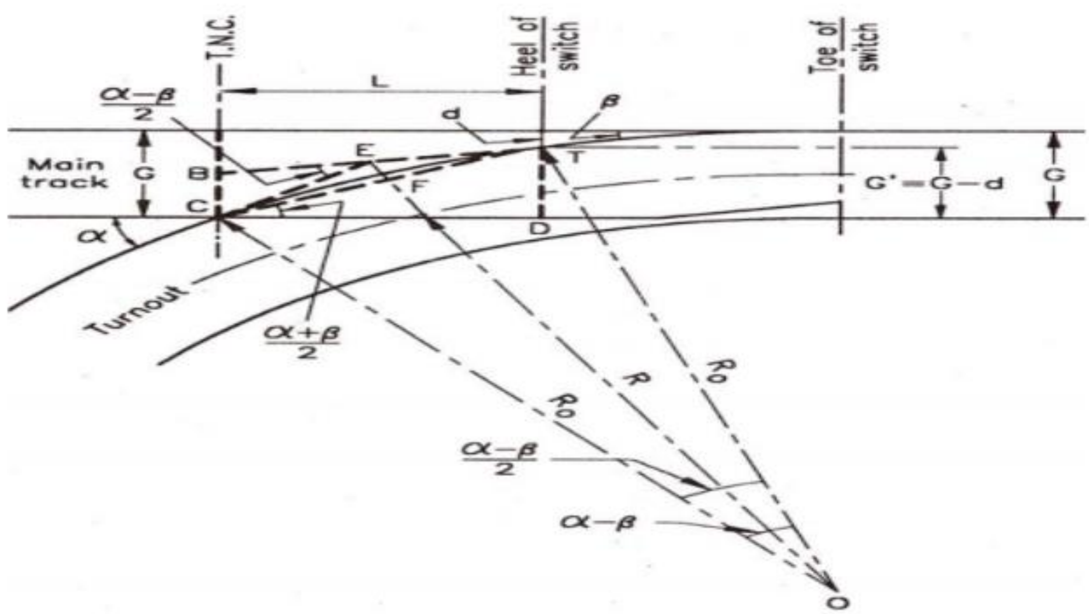


Figure 7-11: Cole's method -2 lead curve

1. Lead or crossing lead (L)

From figure 7-11: triangle TDC

$$L = \frac{(G-d)}{\tan\left(\frac{(\alpha+\beta)}{2}\right)} \tag{7-6}$$

$$L = (G-d)\cot\left(\frac{(\alpha+\beta)}{2}\right)$$

2. R-radius

From triangle OCF

$$Sin(\frac{\alpha-\beta}{2}) = \frac{CF}{Ro} = \frac{CT}{2Ro}$$

$$Sin(\frac{\alpha-\beta}{2}) = \frac{TD}{Sin(\frac{\alpha+\beta}{2})} * \frac{1}{2Ro}$$

$$Sin(\frac{\alpha-\beta}{2}) = \frac{G-D}{2Ro\,Sin(\frac{\alpha+\beta}{2})}$$

$$Ro = \frac{G-D}{2\,Sin(\frac{\alpha+\beta}{2}).\,Sin(\frac{\alpha-\beta}{2})}$$

$$Ro = \frac{G-D}{\cos\beta - \cos\alpha}$$

7-7

$$R = R_o - \frac{G}{2}$$

7.4.3. IRS Method

This method is very similar to method -2 but in this case the straight length at the crossing is provided, so in this method one end of the carve is tangential to the switch heel rail and springs from the heel of the switch and the other end springs from the toe of the crossing and is tangential to the straight length of the crossing. With the use of this method, out of the three kinks, two kinks, namely a kink at the toe of the crossing and a kink at the heel of switch is removed, Only one kink, namely at the nose of the switch. This method is very much suitable where tongue rails and crossing are straight.

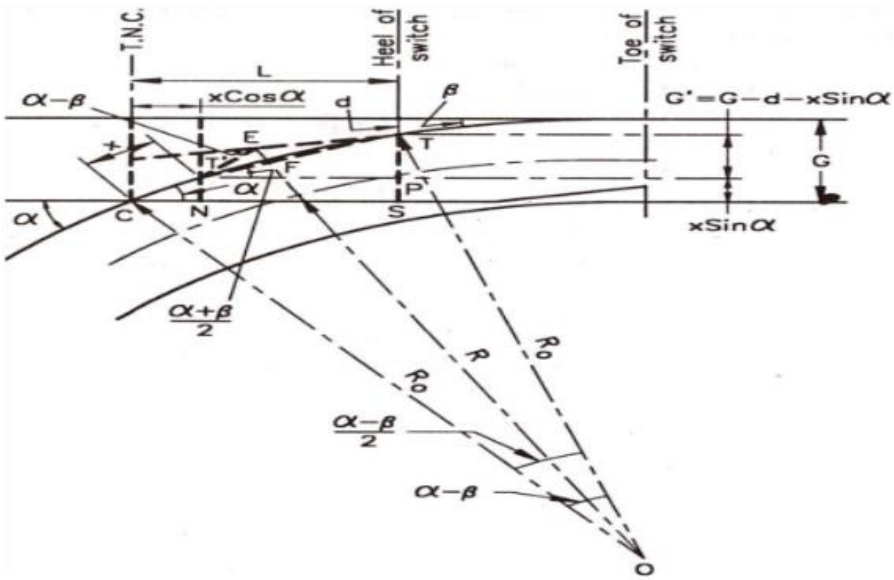


Figure 7-12: IRS method

1. **R-radius**

Now with the given value of G, d, β, α and x the turnout design as follows:

From triangle TPT'

$$TT' = TP\cos ec(\frac{(\alpha+\beta)}{2})$$

$$TF = \frac{TT}{2} = \frac{1}{2TP\cos ec(\frac{(\alpha+\beta)}{2})}$$

From triangle OFT'

$$Ro = TF\cos ec(\frac{\alpha-\beta}{2})$$

$$Ro = \frac{1}{2}(TF\cos ec(\frac{(\alpha+\beta)}{2})).\cos ec(\frac{\alpha-\beta}{2})$$

$$Ro = \frac{TP}{2\sin(\frac{\alpha+\beta}{2}).\sin(\frac{\alpha-\beta}{2})} = \frac{TP}{\cos\beta - \cos\alpha}$$

7-8

$$Ro = \frac{G - d - x\sin\alpha}{\cos\beta - \cos\alpha}$$

$$R = Ro - \frac{G}{2}$$

2. Crossing lead (L)

From triangle CNT' and from line CNS

L=CN + NS =CN + T'P

$$L = x\cos\alpha + G'\cot(\frac{\alpha+\beta}{2})$$

7-9

$$L = x\cos\alpha + (G - d - x\sin\alpha)\cot(\frac{\alpha+\beta}{2})$$

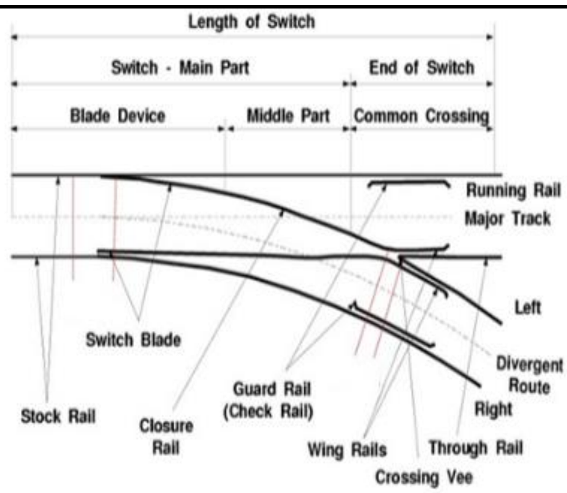
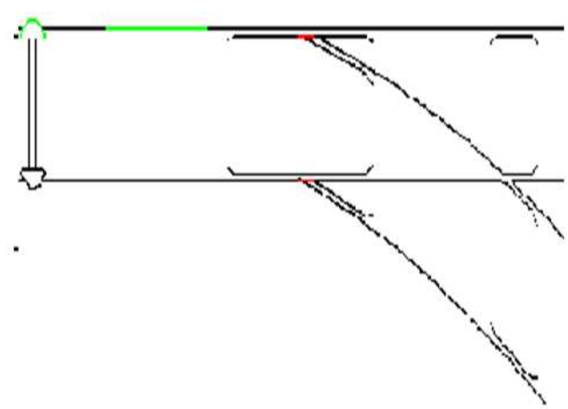
Note: for getting values of method- 2, put x=0 in Equ. 7-8 &7-9 to get:

$$Ro = \frac{G - d}{\cos\beta - \cos\alpha}$$

$$L = (G - d)\cot(\frac{\alpha+\beta}{2})$$

7.4.4. Improved Switch (ISW) Turnout Design

Table 7-1: Basic differences between existing railways switch and improved switch (Terefe, 2017)

EXISTING RAILWAY SWITCH	IMPROVED RAILWAY SWITCH
Use a movable pair of switch (tongue rail)for couching the vehicle wheel	Use fixed switch rail but, there are two types of locker rails, for guarding vehicle wheel for proper position.
Guide the vehicle wheel by moving to either direction of the stock rail: if the tongue rail connects to the left stock rail, the left vehicle wheel will be guided and changed its direction to the right. the reverse is true for the right one	The structural system are designed according to the flange way clearance ,that keeps the fixed passing flange distance so, this makes the wheel flange move on either direction depending on the driver interest
The frog (nose) used in the crossing part and the two side of the frog design for loading	But in this case, switch nose used in switching part, design as, one of the side only for loading and the other side act as wing rail
Most of the time the switch rail manufacture in the industries, by pre casted form	It is possible, to assemble the switch on site by using welding and grinder machine
	

To do the sample of improved switch turnout design, it should be better fixing the value of heel divergence, because of including heel divergent the unknown variable become three (d, SL and CL) for this reason, it should be better developing one equation from the relation of crossing angle and switch angle by given value of heel divergent. The average value of crossing angle and switch angle is constant therefore the switch angle becomes approximately 1/6.4 of the crossing angle ($K = \alpha/\beta = 6.4$) (Terefe, 2017)

Therefore from this relation, one of the unknown variables is identified from the three unknown variable and simplifies the equations from indeterminate to determinate.

Example 7-1: *design geometry of turnout with the following given conditions*

- *Take turnout number seven or 7# (1 in 7 turnout) for 50kg/m steel track with standards gauge of 1435mm 50kg/m rail section having width of head rail=70mm*
- *Consider 0.3 cm for wear; and 6.7cm for flange way clearance according to IRS*

<u>Solution</u>

Step-1: Curve lead

CL= 2GN …. method-1

*CL=2*1.435m*7*

CL=20.09m

Step-2: Radius –R

Ro=1.5 G + 2GN^2 *…… method-1*

*Ro=1.5*1.435 + 2*1.435*7*^{2}

Ro=142.78m

R= Ro-G/2………… method-1

R=142.78-(1.435/2)

R=142.065m

Step-3: Heel divergence (d)

Heel divergent is equal to flange way clearance plus tolerance for the wear plus switch heel (x distance) plus two times the width of head of rail

$d=Fc + w + 2Wh + x$

$d=6.7cm+0.3cm+(2*7cm)+x$

$d=21cm+x$

$d=0.21m+x$

Therefore the heel of the switch is located at the point where the offset of the curve is equal to the heel divergence

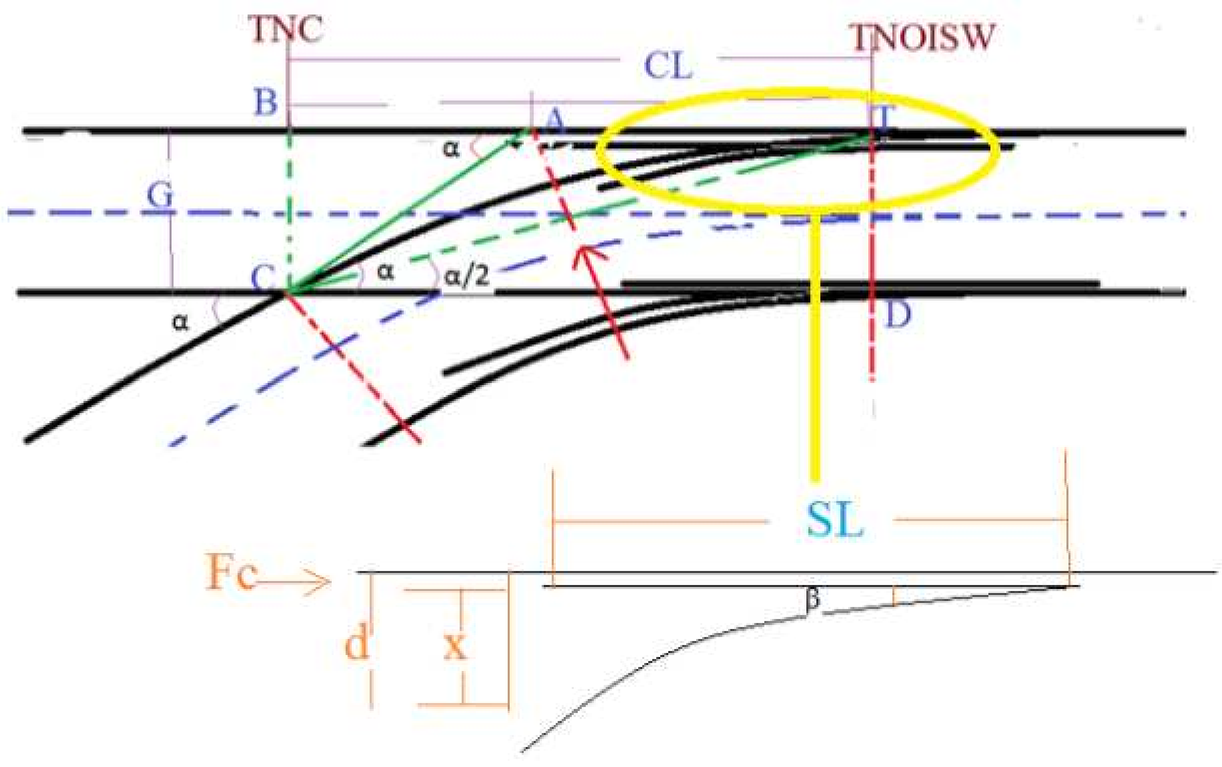


Figure 7-13: Improved switch heel divergent

Step-4: switch lead (SL)

$$SL = \sqrt{2R_o * d}$$

$$SL = \sqrt{2 * 142.78 * (0.21m + x)}$$

$$SL^2 = 59.97 + 285.56x.........eq(*)$$

Step-5: Angle relation constant (K)

the average value of K= α/β=6.4

N=7…………(given sample turnout number)

N=cot α

α= 8.130

β=1/6.4(8.130) β=1.270

$$\tan\beta = \frac{x}{SL}$$

$$\tan 1.270 = \frac{x}{SL} = 0.0222$$

$$SL = 45x..................eq(**)$$

Finally insert equation (**) in to equation ( ), than you will get*

$2025x^2 - 285.56x - 59.97=0$

by using quadratic function, X=0.26m ……………………….approximate positive value

$SL^2=59.97+285.56(0.26m)$

SL=11.58m

d=0.21m+x

d=0.21m +0.26m

d=0.47m

Step-6: lead (L)

$$L = 2GN - \sqrt{2R_o * d} \ldots\ldots method - 1$$

L=CL-SL

L=20.09m – 11.58m

L=8.51m

Note: By using of fixed value of " x " taken from the above section that is x=0.26m and for standard gauge 1435mm with 50kg/m rail section, it is possible to compute for design the rest turnout numbers.

EXERCISE 7-1:

1. Discuss the difference in layouts of turnouts between tramways and regional terminal stations?
2. Which measures, in the planning phase, shall be taken to minimize maintenance cost of switches and crossings?
3. What are the factors to be considered in selection of turnout layout types?
4. Discuss on different operation mechanisms of switches?
5. Prove the switch angle and crossing angle relation (K= α/β=6.4) with following data:
   - For standard gauge 1435 mm from china standard(G=1.435m)
   - 50kg/m rail section
   - Number of crossing calculated by right angle method (cole's method)
   - d= tolerance for the wear + flange way clearance + rail head width (IRS standard) d=0.3cm+6.7cm+7cm d=0.14m

MODULE-EIGHT

8. RAILWAY STATIONS

Introduction

Station is an announcing post with at least one subsidiary track beside the main track. It enables trains crossing (when coming from opposite directions), overtaking as well as starting and finishing of the travel. It is a place on a railway line where traffic is booked and dealt with and where trains are given the authority to proceed forward.

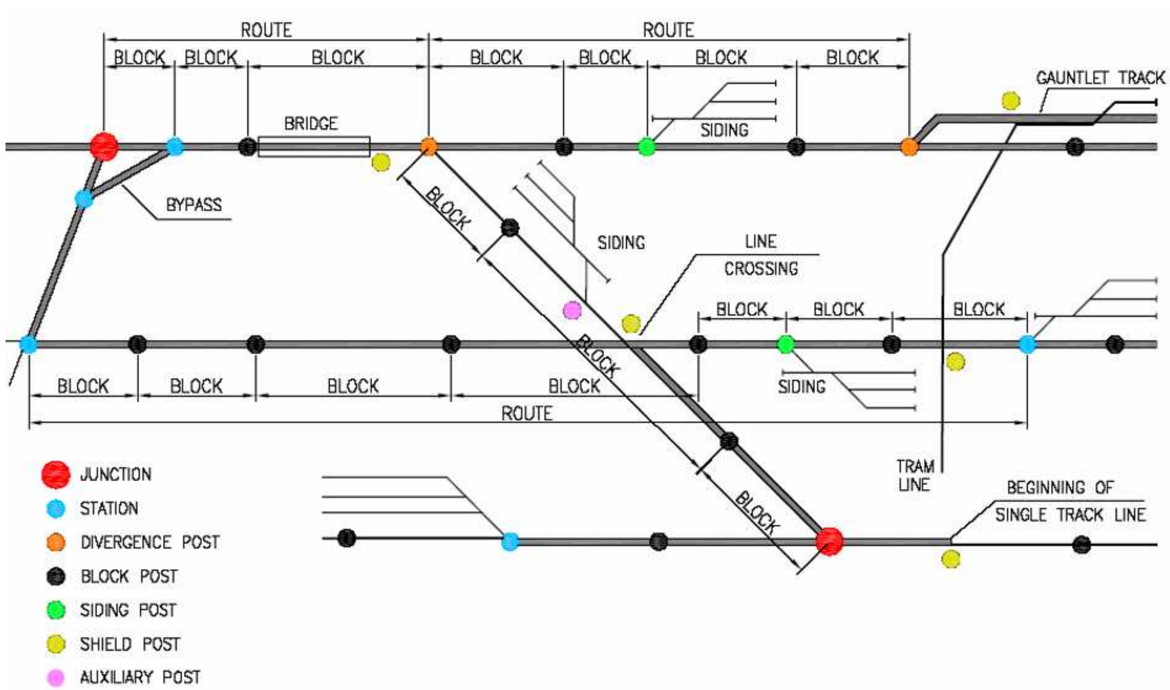


Figure 8-1: railway posts

Stations on railway systems vary enormously in regard to their complexity, suitability and effectiveness but all, in one way or another, will have a direct bearing on the general wellbeing of the final customer, the passenger. Ideally, the customer's requirements should be set down in a brief prepared by the railway operator or controlling authority. These requirements will need to be interpreted into operational guidelines which should form the basis of detailed designs.

On any railway system there is a need to establish uniform standards for the design of stations. This applies to any system but is particularly relevant where new lines or stations are being constructed on existing systems. There is little point in providing new stations with much higher standards than on the remainder of the railway unless there is some chance of adopting these standards in the long term on existing stations.

8.1. Station Planning

In planning any station the following objectives need to be kept very much in mind:

- Attractiveness in appearance.
- Free movement of passengers.
- Safe evacuation in emergency.
- Access for the disabled.
- Access for emergency services.
- Safe accumulation and dispersal of crowds.
- Reliable operation of train service.
- Resilience to failure.
- Cost-eff ective investment.

It is obvious that these objectives cannot be achieved by provision of adequate space alone. A successful station is the product of:

a) well-designed infrastructure,
b) information and signing systems appropriate for the purpose, and
c) A clear well promulgated management philosophy.

A successful railway system will only result from a clear understanding of the interaction between the train service and the stations it serves, both in normal and abnormal operating conditions.

8.2. Site for a Railway Station

The following factors are considered when selecting a site for a railway station.

- Adequate land for the station building, including for any future expansion.
- Level area with good drainage
- The station site should preferably have a straight alignment so that the various signals are clearly visible.
- The site should be near villages and towns; and should be connected by means of approach roads for the convenience of passengers
- There should have adequate water supply for passengers and operational needs.

8.3. Station Classifications

Railway stations can be classified on the basis of various considerations; for instance:

1) ***According to the character of technical operation***
    - In passenger traffic line: the overtaking station, the intermediate station, and the originating train departure-arrival station
    - In mixed passenger and freight railway: crossing station, overtaking station, the intermediate station, district station and marshaling station
2) ***According to the passenger and freight traffic volume***
    - In passenger traffic line: super-large station, large station, medium station, and small-sized station
    - In mixed passenger and freight railway : super class station, Class-I, II, III, IV and V stations (Chinese manual)

8.3.1. Passing Station and Overtaking Station

Passing stations: are set on the passenger-and-freight single-track railway to enable enables trains crossing when running from opposite directions

There shall be two arrival and departure tracks in a passing station; in case of fewer train flow, it may set one track, but not in a continuous manner. Where there is only one arrival and departure track in a passing station, it should be arranged on the opposite side of operation office for train receiving-departure.

Overtaking Station: are set on double track railways and responsible for dealing with the surpassing of trains travelling in the same direction is called overtaking stations. It is to deal with the passing-through of the trains on the main line, entering and exiting the arrival-departure track of waiting trains and stop-and-waiting of trains. In passenger-and-freight railway, it may deal with shunting of trains in opposite directions

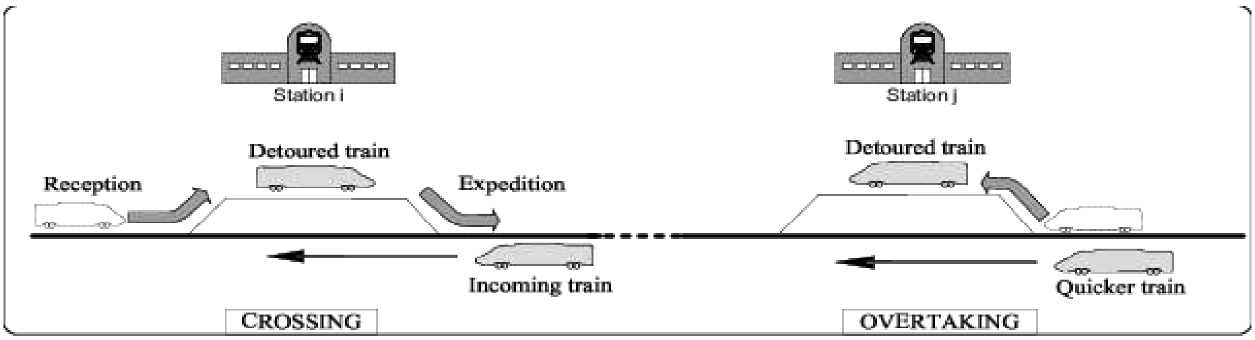


Figure 8-2: Crossing Vs. overtaking station layouts

8.3.2. Intermediate Stations

Distribution of intermediate stations in mixed line for passenger and freight traffic takes into account: the average daily volume and in combination with other transport ways in local areas, and coordination with the urban or regional program. Topographical, geological, hydrographic and railway operation conditions shall be taken in to consideration

Distribution of intermediate stations in high-speed line for passenger traffic:

- The distance between two stations of high-speed railway with passenger transportation business is mainly constrained by the city layout and distance between cities.
- it's necessary to set up an overtaking station between passenger stations with long-distance for high-speed railway to keep the distance between stations balanced

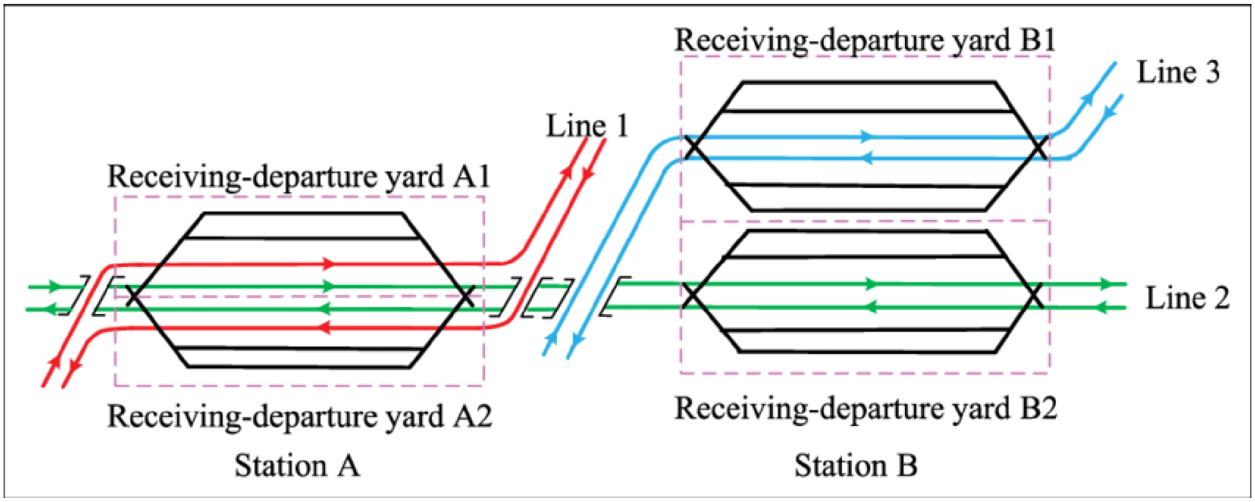


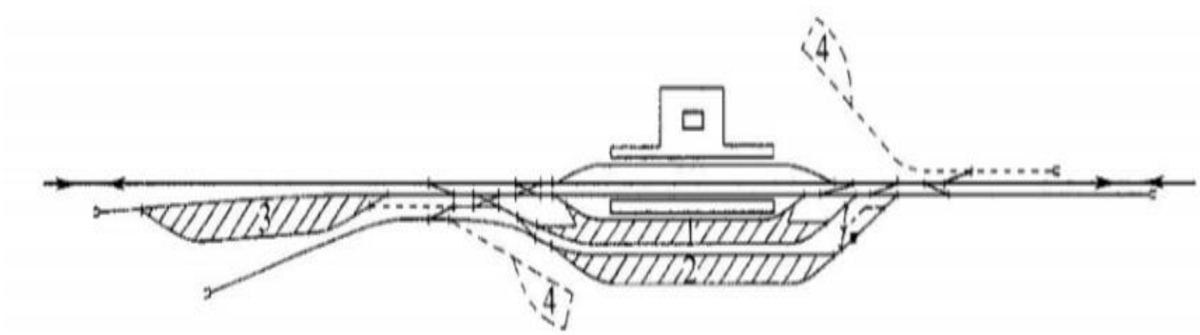
Figure 8-3: Track layout of intermediate stations in China

8.3.3. District Station

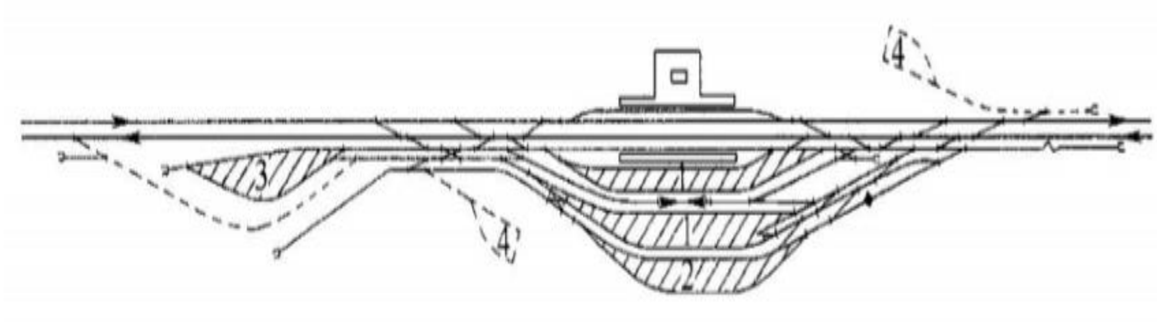
District station is one kind of important technical operation stations on mixed passenger and freight railway; and its layout diagram shall be selected in considerations of:

- number of tracks introduced,
- freight volume, transport nature,
- characteristic of operations in stations and
- routing of passenger/freight locomotives;

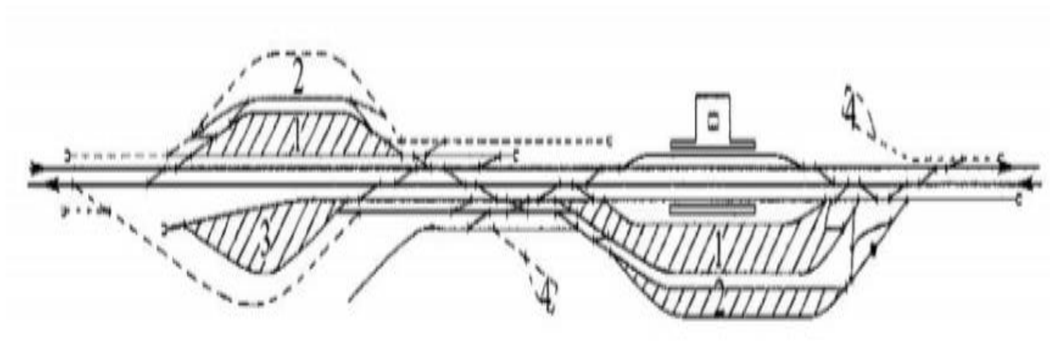
a. Layout Diagrams of Lateral Type for District Station in Single Line Railway



b. Layout Diagrams of Lateral Type for District Station in Double Line Railway



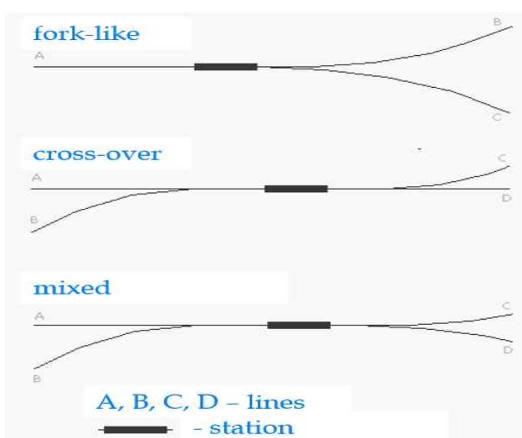
c. Layout Diagrams of Longitudinal Type for District Station in Double Line Railway



Where: 1-Arrival & departure yard; 2-Shunting yard; 3-Locomotive depot; 4-Freight yard schemel

8.3.4. Junction Stations

A station in a crossing of at least 3 lines; like the following layouts



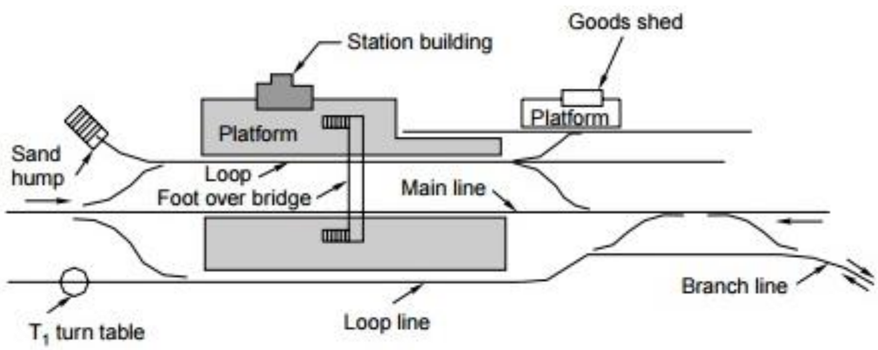


Figure 8-4: junction station with single mainline and single branch line

8.3.5. Marshaling Stations

Its main task is combining of freight trains from marshalled, selected cars coming from various directions or delivered from local loading points and intended to go together to the prescribed destination.

1 Basic duties of the marshalling yards are:
2 Reception of the train.
3 Preparation to marshalling.
4 Cutting the train on selected car groups and directing on separate tracks according to the intended direction.
5 Accumulation of the cars coming from various trains.
6 Putting the cars in order and combination of new trains.
7 Sending the trains from the departure group to the destination stations.

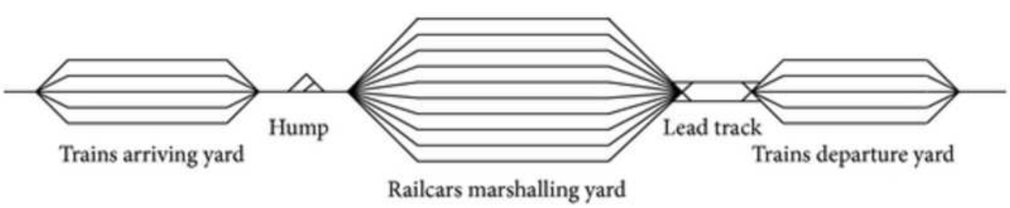
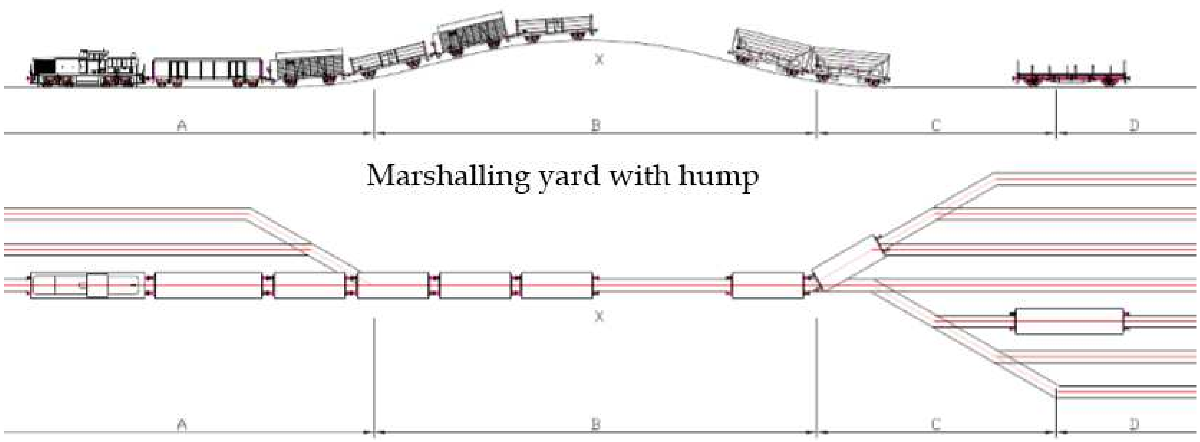


Figure 8-5: The layout of a typical marshalling station



8.3.6. Terminal Station

The station at which a railway line or one of its branches terminates is known as a terminal station or a terminal junction.

- The reception line terminates in a dead end and there is provision for the engine of an incoming train to turn around and move from the front to the rear of the train at such a station.
- a terminal station may need to be equipped with facilities for watering, cleaning, coaling, fuelling, and stabling the engines; storing, inspecting, washing and charging the carriages; and such other works.

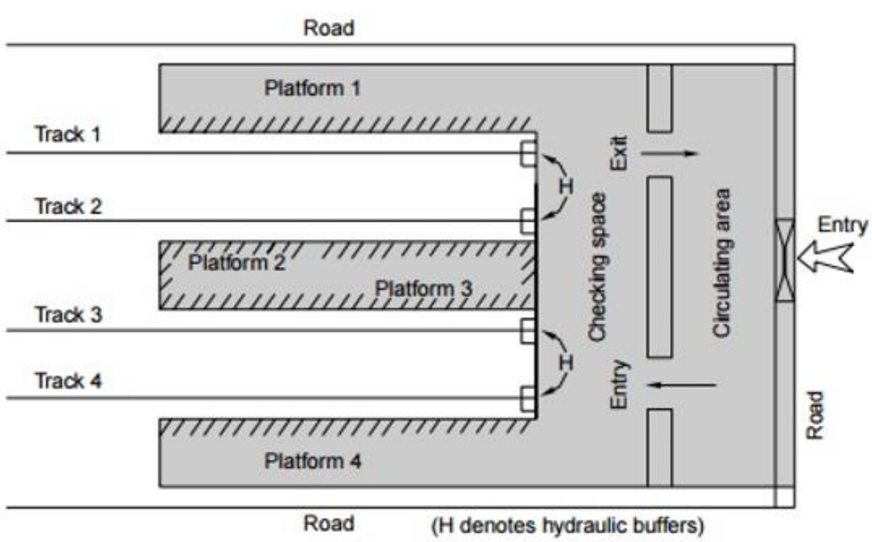


Figure 8-6: layout of a big terminal station

8.4. Facilities Required at Railway Stations

The facilities required at stations are broadly classified into the following main groups.

a. ***Passenger requirements:*** includes waiting rooms and retiring rooms, refreshment rooms and tea stalls, enquiry and reservation offices, bathrooms and toilets, drinking water supply, platform and platform sheds, and approach roads
b. ***Traffic requirements:*** includes goods sheds and platforms, station buildings, station master's office and other offices, signal and signal cabins, reception and departure lines and sidings, arrangements for dealing with broken-down trains, and station equipment.
c. ***Locomotive, carriage, and wagon requirements:*** includes the locomotive shed, watering or fuelling facilities, turntable, inspection pits, ashpits, and ashtrays.
d. ***Staff requirements:*** includes rest houses for officers and staff, running rooms for guards and drivers, and staff canteens.

8.5. Equipment at Railway Stations

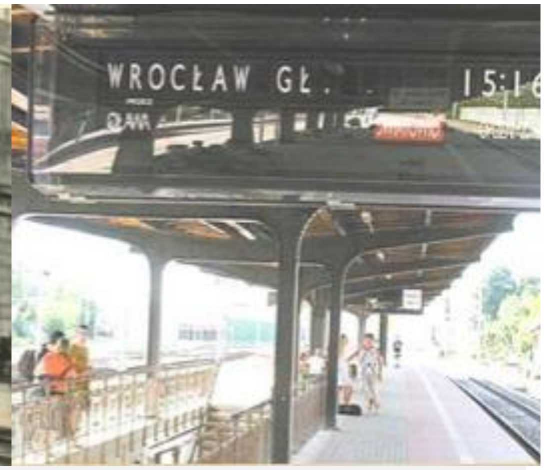
Many different types of equipment are required at railway stations and yards for the efficient working of the railway system. These serve the following purposes.

1) ***Providing facilities for the convenience of passengers:*** platforms, foot bridges, and subways
2) ***Receipt and dispatch of goods traffic:*** Cranes, weigh bridges, loading gauges, and end loading ramps
3) ***Equipment for locomotives and coaches:*** locomotive sheds, examination pits, ashpits, water columns, turntables, and triangles
4) ***Isolation of running lines:*** derailing switch, scotch block, sand humps, buffer stops, and fouling marks.

8.5.1. Passenger Station Equipment

Station building

Audio-visual information system



Underpass and elevator

Grade crossing

Footbridge

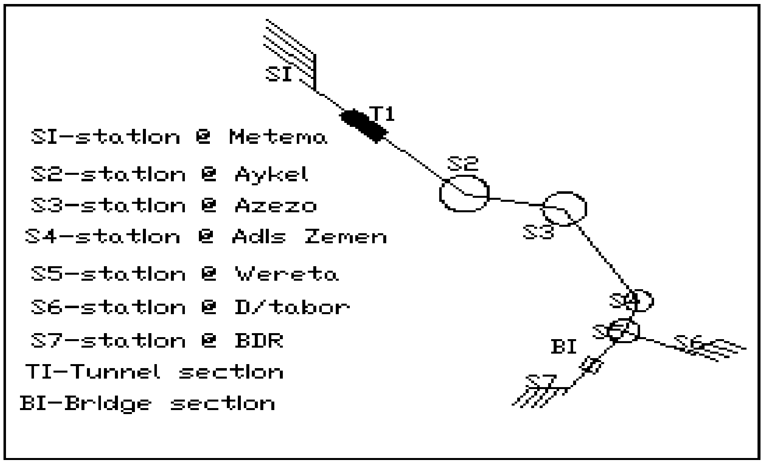
Platforms and shelters

1.1.1. Freight Station Equipment

Warehouse	*Loading ramp*
	
Loading yard	*Gantry crane*
	
Attempt road	*Container transporters*
	

EXERCISE 8-1:

1. List announcing posts and discuss their importance in railroad system?
2. Why establishment of uniform standards for station design is needed for rail transportation?
3. A network, shown in figure below, is designed as single track having bottleneck section from Wereta-Bahir-Dar with estimated theoretical capacity of 39 trains per day, considering 75% desired utilization. Propose a station type along with its equipment's for S_1, S_3, S_4, S_5, S_7? And why?



Station	**S_1**	**S_2**	**S_3**	**S_4**	**S_5**	**S_6**	**S_7**
Inhabitants	6,581	24,146	227,044	26,849	44,560	60,596	200,174

4. Discuss how the minimum length of tracks in passing and overtaking station is determined?
5. Suppose you are part of station 'design team'; discuss on the parameters for selecting best marshaling station layout?
6. List most common station facilities for freight vs. passenger station?

MODULE-NINE

9. TRAIN OPERATION

Introduction

Railways are complex technical systems. They are often considered to be static, stiff and inflexible. As long as only constant, non-dynamic parts, such as the infrastructure, are considered, the system is quite easy to understand. But the reality for train operation system is different. The variance in different parameters makes the system difficult. For instance:

- The infrastructure is adjusted and complemented all the time. Most of the adjustments are minor, but these changes nonetheless imply that important factors for operation vary over time.
- The timetable creates a well-defined structure. However, the capacity is utilized differently every day since the actual timetable varies from one day to another due to delays, extra trains and cancelled trains. Therefore the planned timetable is modified from time to time. The principles for this capacity allocation also change over time.
- The available capacity, which is an important condition for the timetable, varies over time. Failures, construction works, accidents and delays all make the available capacity vary over time.
- The railway system is used and operated by humans. Human behavior varies naturally from one time to another. Train crews, dispatchers and passengers all contribute to this variance.

Definition of important terms

a) ***Train number:*** particular number which is labeled to each train in order to identify the nature and grade of the train, and to facilitate the train organization and management.

b) ***Car flow (wagon flow):*** the collections of vehicles with some destination by railway transport.

c) ***Train flow:*** the collection of trains with some destination

d) ***formulation plan:*** the plan of converting the wagon flow into train flow

e) ***wagon (car) flow organization:*** Converting the car flow into train flow

f) ***Train diagram:*** indicate the range of operating trains and in railway stations or through the time to send the technical file, which provides train occupies the interval procedures, train arrival and departure of each station (or through) time, the train in the run-time interval, the train stops at the station and the locomotive routing time, weight and length of trains, train the whole way the basis of the organization

g) ***Shunting operation plan:*** is the specific action plans which explain how to break up, marshal, receive or send or pick-up and drop trains for shunting groups. Shunting operation should be done based on the shunting operation plan.

h) ***Railway transportation production plan:*** is divided into long-term plans, annual plans and monthly plans according to its preparation period; and is made up of month Railway goods transportation plan and Railway transportation technical plan.

9.1. Operational Areas of Railway Station

There are four main functional areas typically housed in most stations as a general. They are; core, transition, peripheral, and administrative areas. The main users of these four functional areas are generally passengers, staff, guest and visitors

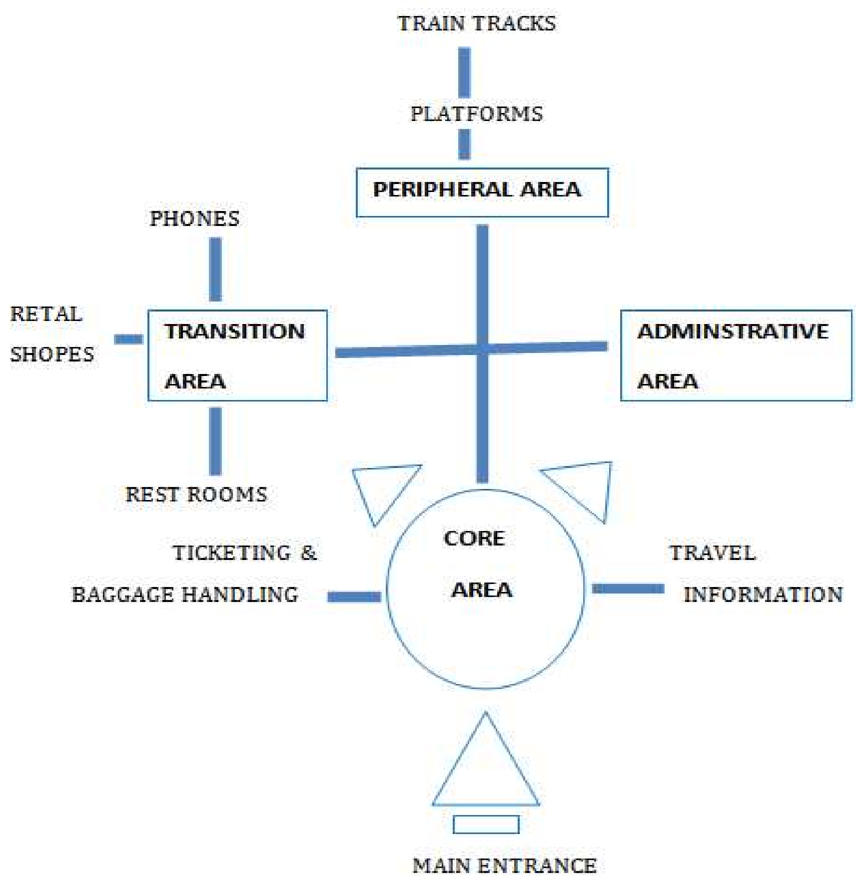


Figure 9-1: Operational areas of railway station

I. ***Core areas:*** focus on processing passengers. It includes ticketing, information, baggage handling, reclaiming, and waiting.

II. ***Transit areas:*** connect transit facilities in the core areas to the transportation modes. It includes restrooms, telephones, and commercial spaces.

III. ***Peripheral areas:*** support circulation outside the main buildings. They often include platforms, tracks, and vehicle service spaces.

IV. ***Administrative area:*** control both traffic and station management. These areas can be isolated from other facilities or inserted among them.

9.2. Single and Double Track Operations

Infrastructure configuration, timetable design and delays play important roles in the competitiveness of railway transport operation. This is especially true on single-track lines where the run times and other timetable related parameters are severely restricted by crossings (train meetings). The crossings also make the lines' operation more sensitive to disturbances.

Double-track lines operated with mixed traffic show properties similar to those of single-tracks. In this case overtaking implies scheduled delays as well as risk of delay propagation.

9.3. Train Schedule

The train schedule is the backbone of rail transport planning and operation, i.e. the generation of train schedules is the core subject for train operation. A train schedule consists of the arrival and departure times of the lines at certain points of the network. In general, schedules for rail transport are periodical, i.e. the schedule is repeated after a basic time period or, for short period. In designing a schedule the following things must be taken in mind:

a) Appropriate allocation of rolling stock units to the trips to be operated.

The trips established by the train schedule must be performed by some vehicles (motor unit, coaches) and crews (like engine drivers, conductors etc.). Relevant objectives to be pursued are service to the passengers, efficiency, and robustness. Service to the passengers' means that on each trip the provided capacity should be sufficient to transport the expected numbers of passengers according to given norms.

b) Productivity of crew

The practical capacity of railway line is directly proportional to crew productivity. Regular rules and training sessions should be provided to ensure that the qualification of operating crews is maintained and to expand the operating personnel pool. There should seek uniformity across all train operations, so that training is simplified, operating personnel are entirely interchangeable within various tasks, and misunderstandings about procedure are minimized.

9.4. Capacity of Railway Line

According to International Union of Railways (UIC), Railway Capacity is:

"The total number of possible train paths in a defined time window, considering the actual path mix or known developments, respectively"

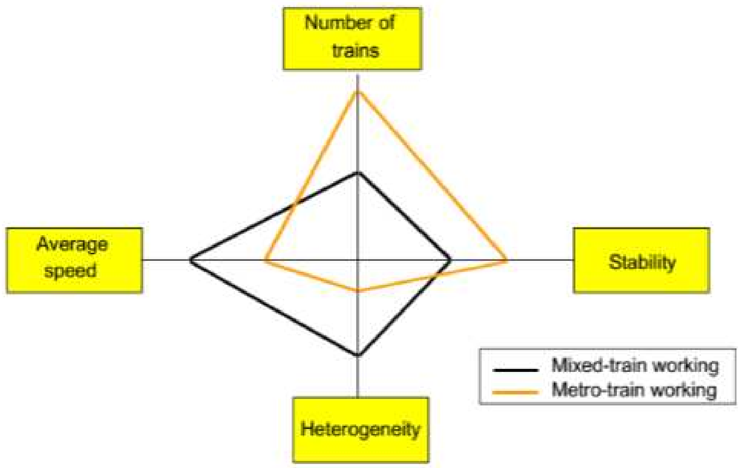


Figure 9-2: Capacity balance (UIC – 406)

1.3.1. Types of Railway Capacity

I. **Theoretical Capacity**

It is the absolute maximum capacity which can be achieved subject to absolute train-path harmony (the same parameters for majority of trains); and minimum headway (shortest possible spacing for all trains). It is almost impossible to achieve theoretical capacity in practice.

II. Practical Capacity:

It is the practical limit of "representative" traffic volume that can be moved on a line at a reasonable level of reliability. The "representative" traffic reflects the actual train mix, priorities, traffic bunching, etc.

Table 9-1: The ratio which proportionate practical and theoretical capacity according to UIC405

Type of line	Peak hour	Daily period	Comment
Dedicated suburban passenger traffic	85%	70%	The possibility to cancel some services allows for high levels of capacity utilization.
Dedicated high speed line	75%	60%	Can be higher when number of trains is low (smaller than 5 per hour) with strong homogeneity.
Mixed-traffic lines	75%	60%	Can be higher when number of trains is low (smaller than 5 per hour) with strong heterogeneity.

Each railway system needs to select a design density for peak periods, which will reasonably allow for minor incidents, delays and up to say three cancellations. It is essential in capacity planning of stations to off er sufficient resilience to train service perturbations and surges in demand that staff intervention by station control becomes the exception rather than the rule.

9.5. Methods to Evaluate Railway Capacity

9.5.1. Analytical Methods

It uses several steps of data processing through mathematical equations or algebraic expressions.

a. <u>UIC Leaflet 405</u>

$$C = \frac{T}{t_{fm} + t_{zu} + t_r} \tag{9-1}$$

$$t_r = \begin{cases} 0.67 * t_{fm}, \text{when the desired utilization} = 60\% \\ 0.33 * t_{fm}, \text{when the desired utilization} = 75\% \end{cases} \tag{9-2}$$

$$t_{fm} = \frac{\sum n_i * n_j * t_{fij}}{\sum n_i * n_j} (\min / train) \tag{9-3}$$

Where:

- *C=capacity of a section ,in number of trains, in period T;*
- *t_{zu}= additional time, based on the number of sections*
- *t_{zu} = 0.25*a. \`a\` is the number of block sections*
- *t_r= running time margin which is added to train headways to achieve an acceptable quality of service.*
- *t_{fm} =average of minimum train headways*
- *t_{fij} =the minimum headway b/n a trains of type **j** following trains of type **i**, (min/train)*
- *n_i =number of train of type **i***
- *n_j = number of train of type **j***

Note: the time table is required to know the sequence of case where one train of type ***j*** follows one train of type ***i*.**

Example 9-1:

Given conditions

- Double track section from A to B length 18.309km.
- Take block sections of fixed length 3 km
- the desired utilization is 75%
- minimum train headway (from time table)=11min

Solution:

Step-1: average of minimum train headways, t_{fm}

$$t_{fm} = \frac{\sum n_i * n_j * t_{fij}}{\sum n_i * n_j} (\text{min}/\,train)$$

$$= \frac{1*1*11+1*2*11+2*1*11+1*1*11+1*2*11}{1*1+1*2+2*1+1*1+1*2} = 11\,\text{min}/\,train$$

Step-2: running time margin, t_r

$t_r = 0.33 \times t_{fm}$, for the desired utilization of 75%

$t_r = 0.33 \times 11 = 3.63$

Step-3: an extra time, t_{zu}

$t_{zu} = 0.25 * a$, where, $a = 18.309 km / 3 km = 6\, block\ sections$

$t_{zu} = 0.25 * 6 = 1.5$

Step-4: Theoretical capacity, C

$$C = \frac{T}{t_{fm} + t_{zu} + t_r}$$

$$C = \left(\frac{1440 \min/day}{(11 + 1.5 + 3.63)\min/train}\right) = 89\, trains/day$$

b. <u>UIC Leaflet 406</u>

To evaluate the railway capacity according to the UIC 406 method, the railway network has to be divided into line sections. For each line section the timetable has to be compressed so that the minimum headway time between the trains is achieved.

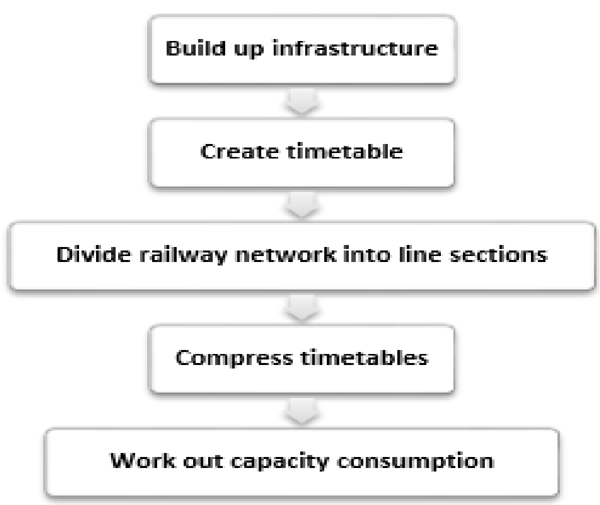
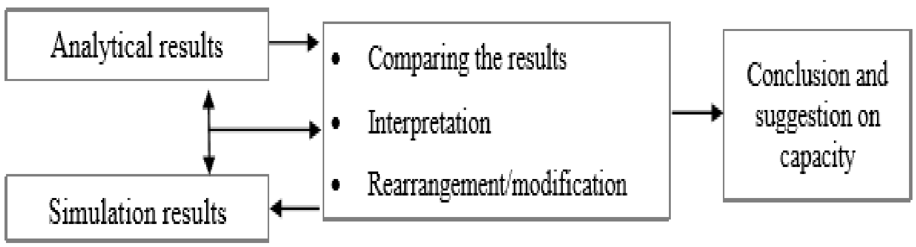


Figure 9-3: Workflow of the UIC-406 method

9.5.2. Simulation Methods

Simulation is an imitation of a system's operation which should be as close as possible to its real-world equivalent. Common software used in this category are: Multi-Rail (U.S), RAILSIM (U.S), Open-Track (Switzerland), SIMONE (Netherland), Rail-Sys (Germany), DEMIURGE (France), RAILCAP (Belgium), and CMS (UK)

9.5.3. Combined Analytical-Simulation Approach



9.6. The pairs of trains

The pairs of trains which will operate per day to satisfy the forecasted demand is calculated by dividing the volume of Demand per day by the maximum possible hauling capacity of one set of train. In doing so, length of train is governed by the maximum length of platform of the station.

Example 9-2: *Calculate the number of daily passenger trains with the following given conditions*

Figure 9-4: One set of freight train

Table 9-2: Types of freight wagons which are ready for operation

type	Box Wagon	Uncovered Gondola wagon	Covered Gondola Wagon	Hoper Uncovered wagon	Hoper Covered wagon	Flat Wagon	Refrigerated wagon	Tank wagon	Center Beam wagon	Bi-level wagon
number	220	20	110	20	20	550	10	110	20	20
Capacity (ton)	70	70	70	70	69	70	38	70	70	22
Length (m)	16	23	23	23	23	23	16	18	23	25

Solution:

Considering 800m total length of wagons, the number of wagons for this length is calculated by weighted average by taking the parameters from table above

Step-1: weighted average length of one wagon (table above)

=21.07m;

Step-2: weighted average weight which can be carried by one wagon (table above)

=68.82ton;

Step-3: the number of wagon will become;

=800m/21.07m = 37 wagons can be coupled at a time.

Step-4: Total amount of average freight volume which can be transported by one set of train

=37wagon*68.82ton = 2546.34ton.

Step-5: The number of daily trains

= (Forecasted annual freight vol.)/ (2546.34ton*365days)

EXERCISE 9-1:

1. Which things make passenger transport operation system in different with the corresponding activities of freight transport?
2. Why achieving theoretical capacity is almost impossible in practice?
3. Discuss on the procedures to be followed for analyzing railway demand in both passenger and freight traffic?
4. How to come up with capacity improvement need of a given existing railway network?
5. Calculate the number of daily passenger trains with the following given conditions

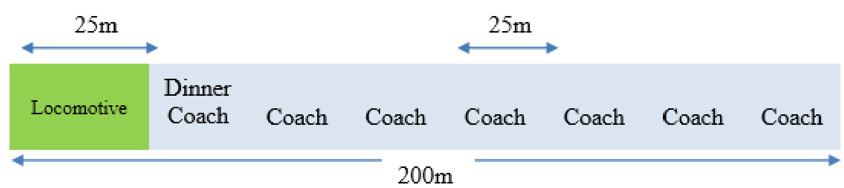


Table 0-1: types of passenger coaches

Type	hard seat coaches	Hard Sleeper coach	Soft Berth coach	Dinner Coaches
number	20	4	4	2
Capacity(passenger)	118	66	36	50
Length (m)	25	25	25	25

6. Discuss the impact of the following factors towards better operation of railway line?
   a) Railway demand
   b) Crew (personnel)
   c) Rolling stock
   d) Train schedule
   e) Railway network

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APPENDIX

Semester Project

Instructions

- *Form a group of five and name with a title that advocates love and peace for, our beloved country, Ethiopia.*
- *Select two points from Ethiopian national railway network, not exceeding 50km, for which you can have necessary data's to undertake tasks related to your semester project.*
- *It's better for, a line to be selected, having a bridge and tunnel sections*
- *Your semester project should have, but not limited to, the following main contents*

1. **Preliminary Design of Railway Line**
   1.1. Environmental impact assessment (EIA)
   1.2.Feasibility study of railway line
2. **Design of Railway Track Structures**
   2.1. Selection of track type
   2.2. Detail design and selection of track components
3. **Selection of Tunnel Excavation Equipment**
   3.1. Description of project characteristics
   3.2. Selection methods
4. **Bridge Loading and Design**
   4.1. Selection of bridge type
   4.2. Layout of proposed bridge
   4.3. Bridge floor design
5. **Operation of Railway Line**
   5.1. Selection of station types and its distribution
   5.2. Description of proposed train control system
   5.3. Train schedule

All Tracks Advance to Love and Peace!