ADDIS ABABA INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

STANDARDIZATION OF GUIDELINES FOR RAILWAY TRACK INFRASTRUCTURE SUBSYSTEM FOR RAILWAY SYSTEM OF ETHIOPIA

By

Yeserah Gebeyehu

A thesis submitted to the school of Graduate Studies of Addis Ababa University in Partial fulfillment of the degree of Master of Science in Civil Engineering

Advisor

Berhanu Abesha (Dr.–Ing.)

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Addis Ababa Institute of Technology [**AAiT**]

Approved by Board of Examiners

Berhanu Abesha (Dr. -Ing.)	________________	______________
Thesis Advisor	Signature	Date
Bikila Teklu (Dr.)	______________	______________
Internal Examiner	Signature	Date
Debo Tunka (Ato)	______________	______________
External Examiner	Signature	Date
Asres Semieneh (Ato)	______________	______________
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Abstract

Railway transportation, which is needed in the achievement of effective development, is an efficient, cost-effective and it is environmental friendly transport system. In addition to the low railway network connectivity in Ethiopia like other developing countries, the available one is not modern; has low capacity and is insufficient to facilitate transit trade; and also Ethiopian railway transport has not standardized guidelines to design the system.

In this study an attempt was made to standardize guidelines for railway track infrastructures specifically track geometric parameters and track structural elements for conventional lines in Ethiopia. To achieve such objectives the research had a methodology of a task involving literature review; discussion and comparison of different country standardization practice; and different data collection that contribute for standardization of guidelines for railway system in Ethiopia.

Standardization of guidelines for different track infrastructure subsystems for Ethiopia context was done based on topography, climatic condition and availability of track construction material, and different literature reviews.

Track cant of a normal limiting value of 160 mm was recommended to reduce or eliminate lateral acceleration created due to small curve in difficult topography of Ethiopia. In addition a cant excess of as small as 70 mm was recommended in avoiding damage of low rail by heavy freight train; and correspondingly this can be achieved by increasing the value of cant deficiency for fast train. A gradient of up to 3.5% and/or greater value was recommended for lines constrained by topography.

Even if the selection of steel rail grade is based on avoiding rail wear and rolling contact fatigue; standard steel rail strength i.e. R260 is recommended for all lines including for the whole range of curvatures due to high investment cost of higher quality steel grade that are recommended for sharp curves (curves are suffering from high rail wear). On the other hand, the recommended method of selecting fastening systems is a matter of satisfying safety of the lines.

The specification of ballast and subballast depend on the availability of track construction material; and the track construction material availability in Ethiopia is, more or less, not a problem in construction industry even in the eastern part of Ethiopia where inadequacy of construction material are expected. An attempt was made to find a suitable range of ballast

gradation which fulfills the objective of good strength (well graded) without significant reduction in permeability of ballast (uniform graded) using a coefficient of uniformity between 2.2 and 2.6, moderately graded ballast was recommended. In addition the specification of subballast material was determined based on filter principle of drainage criteria using subgrade material size distribution. The problematic subgrade soil, placed under subballast, should be replaced or improved to achieve the required strength characteristics of subgrade layers.

The appropriate type of tests was selected to define the characteristics of each track substructure layers (ballast, subballast and subgrade soil).

Key Words: Track, Cant, Cant Deficiency, Cant Excess, Rail, Fastener, Sleeper, Ballast, Subballast, Subgrade and Safety

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Notation

Notation	Definition
	Track Geometry Parameters
S	Center to center length of track gauge
R	Radius
α	Curve angle subscribed by a 100ft
k	Curvature
a_l	Centrifugal lateral acceleration
v	Speed, m/s
V	Speed, km/h
h_t	Track cant height
h_{eq}	Equilibrium cant
h_d	Cant deficiency
h_e	Cant excess
θ	Track cant angle
$\emptyset$	Lateral force angle
g	Gravitational acceleration
a_y	Component of resultant acceleration parallel to the track plane
a_z	Component of resultant acceleration perpendicular to the track plane
L_t	Transition length
(dh_t)/dt	Rate of cant
(dh_d)/dt	Rate of cant deficiency
dh_t/dx	rate of cant as a function of length
	Track Structure Elements
f_{yk}	Tensile strength of steel
f_{yd}	Yield strength of steel
P	Design wheel load
P_s	Static wheel load
$\emptyset$	Dimensionless impact factor
D	Wheel diameter
k	Track modulus
M	Bending moment
E	Modulus of elasticity
I	Moment of inertia

Z_b	Section modulus at base
σ	Bending Stress
q_{mean}	Mean contact stress
q_r	Rail seat load
R_w	wheel radius (m)
υ	poison's ratio ($\upsilon = 0.3$ for steel rail)
τ_{max}	Maximum shear stress
τ_{all}	Permissible shear stress
f_s	Combination of factor of safety
σ_{all}	Allowable bending stress
σ_y	Yield stress
σ_t	Temperature induced stress
DF	Load distribution factor
P_a	Average intensity of pressure on ballast
V_f	Speed factor
T_f	Tonnage factor
f_c'	concrete strength
f_{pu}	Ultimate tensile strength of wire or strand tendons and
f_{py}	Yield tensile strength of wire or strand tendons
M_R=	Moment resistance
Z	Section modulus
P	Pre-compression due to transfer of prestress to concrete
f_r	Modulus of rapture of concrete
C_u	Coefficient of uniformity
C_c	Coefficient of Curvature
P_c	Maximum intensity of pressure on subgrade
h_b	Depth of ballast below tie
h_{sb}	Subballast layer thickness
T_s	Tie spacing
P_v	Vertical load carrying capacity of shoulder
ΔT	Temperature change
BSW	Ballast section shoulder width

1. Introduction

1.1. Background

Transportation structures are very important as significant drive for the growth of both economy and society of any country around the world. In addition these infrastructures play crucial roles in the effort to achieve the goals of poverty eradication and sustainable development. Universally it is also witnessed that both cargo volume and loads are going on increasing from year to year with alarming rate all over the world. Such growth demands better transportation means for efficient and reliable transport of commercial and industrial products without delay.

Railway transportation is one of the important infrastructure facilities that are needed in the achievement of effective development; and provides an efficient, cost-effective and environmental friendly transport system which can quickly haul large volumes of goods which are not easily transported through motor vehicles for long distances [Kaewunruen and Remennikov, 2008; Abii Tsige, 2009].

In developed countries, railway transportation is becoming a reliable and regular form of transport a bulk goods with high speed and certainty. In addition, train travel brings much safety, convenience and comfort to passengers than other means of transportation systems. Whereas in Ethiopia, 95% of freights and 97% of passengers are transported by road transports with 46812 km network of indicated in Appendix C, Figure C-1(of which 6938 are paved road), which have very significant impact on the infrastructure as well as accelerating distress and reducing the durability and performance of the road structure [Ibrahim Worku, 2011].

Even if ambition of development of railway transport is fast, the system faces an obstruction in different factors [CDE, 2007]. General obstructive factors for the development of railway transportation on the existing lines are

- Lack of maintenance,
- Poor management,
- Lack of commercial focus,
- Dominant single line operation,
- Insufficient infrastructure capacity and performance ,
- Tight curves and high gradients,
- Lack of automatic signaling systems on major line sections and insufficient number of skilled and non-skilled staff employed.

- As for freight transportation, low operational speed, lack of sufficient traction power for long hauling, higher operational costs are leading factors of railways confining power to compete with other modes.

In addition to the low railway network connectivity in Ethiopia like other developing countries [CDE, 2007], the existing line(Chemin de fer Djibouto-Ethiopien [CDE]) is not good enough to facilitate transit trade and is insignificant to transport mainly passengers and small parcels of goods in the corridor.

Among the modern types of railway tracks, ballasted railway track is often used for accessing to remote and rural area. The financial viability of the ballasted track relies on its cost-effectiveness in construction, maintenance, and renewal [Kaewunruen and Remennikov, 2008]. The ballasted track structure provision aims to distribute the large and concentrated wheel loads to the track components relatively with uniform fashion [Selig, 2004] and focused primarily on accommodating increased vehicle weights, faster operating speeds, and reduced track maintenance cycles with ensuring system safety, reliability, and profitability as the railroads strive to compete with other transportation modes for the movement of freight and passengers [Smithberger, 2000].

In addition track structures guide and facilitate the safe, cost-effective, and smooth ride of trains [Remennikov, 2008]. The main components of typical ballasted railway track can be subdivided into two main groups: superstructure and substructure. The most obvious components of the track such as the rails, rail pads, concrete sleepers, and fastening systems form a group that is referred to as the superstructure. The substructure system consists of ballast, sub-ballast and subgrade (formation). Both superstructure and substructure are mutually vital in ensuring the safety and comfort of passengers and a satisfactory quality of ride for passenger and freight trains.

Rail track structures are designed with an objective of achieving a minimum standard of capacity and track geometry to ensure safer operation of trains at specified levels of speed, axle load and tonnage to be hauled [Radampola, 2006].

In this research an attempt was made to standardize guidelines for the railway track infrastructure subsystem mainly include track geometry parameters and track structures in Ethiopia of high plateau terrain with central mountain range divided by Great Rift Valley.

1.2. Statement of the Problem

In developing countries like Ethiopia, the rail network connectivity is not only low but also it is not good enough to facilitate transit trade; of course most section of the existing line is more or less it is not functional. There are vast movement of the people, huge import and export of goods which are not efficiently supported by railway transport. In addition to less availability of railway transport in Ethiopia, the available one is not modern; has low capacity and is insufficient to facilitate transit trade; and also Ethiopian railway transport has not standardized guidelines to design the system. So in this study an attempt was made to standardize guidelines for railway track infrastructure specifically track geometric parameters and track structural elements.

1.3. Objective of the research

Since the railway transport is becoming a reliable and regular form of transporting bulk goods and train travel brings much safety, convenience and comfort to passengers than other means of transportation system; this study was done in this sector in order to do further research in the development of modern system of railway transportation in Ethiopia.

The main objective of this research was standardizing guidelines for railway track infrastructure subsystem of railway transport of Ethiopia to produce economically feasible construction, environmental friendly, safe, comfortable and competent mode of transportation system in Ethiopia.

The specific objectives of this study are:

1) To standardize design guidelines for track geometry elements for safe and effective operation in railway transport.
2) To standardize guidelines for design of track structure elements [ballast, subballast, subgrade soil, sleeper, rail and rail fastening system].
3) To raise awareness to safety issue and measures in railway transport.

1.4. Research Methodology

The research methodology for this study involved the following major tasks: literature review; discussion and comparison of different country standardization practice; and different data collection that contribute for standardization of guidelines for railway system in Ethiopia. The types of data collected by document review include topographical data, climatic condition data and type of construction material and availability data. The significant numerical analysis was done with the statistical tool of MS-Excel. Finally conclusions and recommendations for future work are presented.

1.5. Scope of the research

Standardization of guide lines for track infrastructure for railway system of Ethiopia is limited to conventional line that is not including light rail transit (LRT). For this conventional railway line, the study basically concentrates on track geometry parameters and track structural elements including the safety system that concerns on these issues.

2. Literature Review

2.1. General

The railway constituting conventional rail system may be divided into subsystems of either structural areas or operational areas [TSI, 2008]. 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.

In addition TSI (2008) showed that 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.

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 superelevation (transverse vertical plane) [Sadeghi, 2010].

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 [Kaewunruen and Remennikov, 2008]. 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.

Since railway infrastructures are assets which represent a high investment; they are designed to work in very demanding safety conditions and must display a very low occurrence of failures [Simoes, 2008].

2.2. Historical Background of Railway

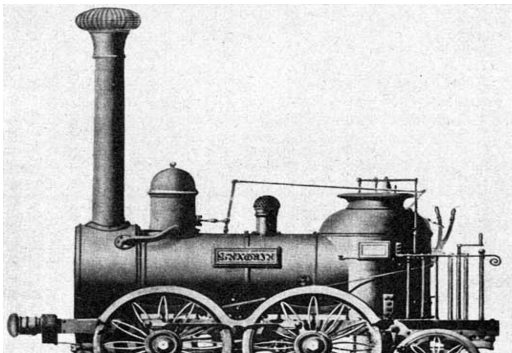
2.2.1. Railway History of the World

Like so much in present day society, the railroad was the result of industrial revolution although the idea of a special track for hauling goods dates back about 2000 years. Railways that fit Lewis's definition, that is "railway is a prepared track which so guides the wheels of the vehicles running on it that they cannot leave the track", existed as far back as the 6th century BC; the Greek Diolkos was a railway with a track made from stone, 6km in length across the Peloponnese, used for transporting ships until the 9th century AD – an extraordinarily long period [Coulls, 1999].

Most historians agree that the opening of the 48km (30 miles) double track line of metal rails of Liverpool-Manchester Railway in 1830 in the north-west of England (Coulls, 1999 and Bonnet, 2005), the world's first true railway, the prototype of the 'modern railway' had arrived: a combination of specialized track, the accommodation of public traffic, the conveyance of passengers as well as freight, mechanical traction, and some measure of public control.

In the second half of the 19th century, the more advanced industrial states engaged in a worldwide contest for strategic advantage, economic fortune and imperial expansion. As industrial empire arose, railways became a means to great power and status. The colonial railways in Africa, Asia and South America were thus an essential part of the spread around the world of the economic processes, ideas, and institutions of the European powers: the production of new foodstuffs and raw materials to feed the industries and peoples of the West, new populations to produce them, new patterns of land ownership, and new legal codes to make the conquered lands safe for investment and exploitation [Coulls, 1999].

(a)



(b)

Figure 2.1: Railway transportation in the early 19th century (a) freight train (b) steam locomotive [David, 2006]

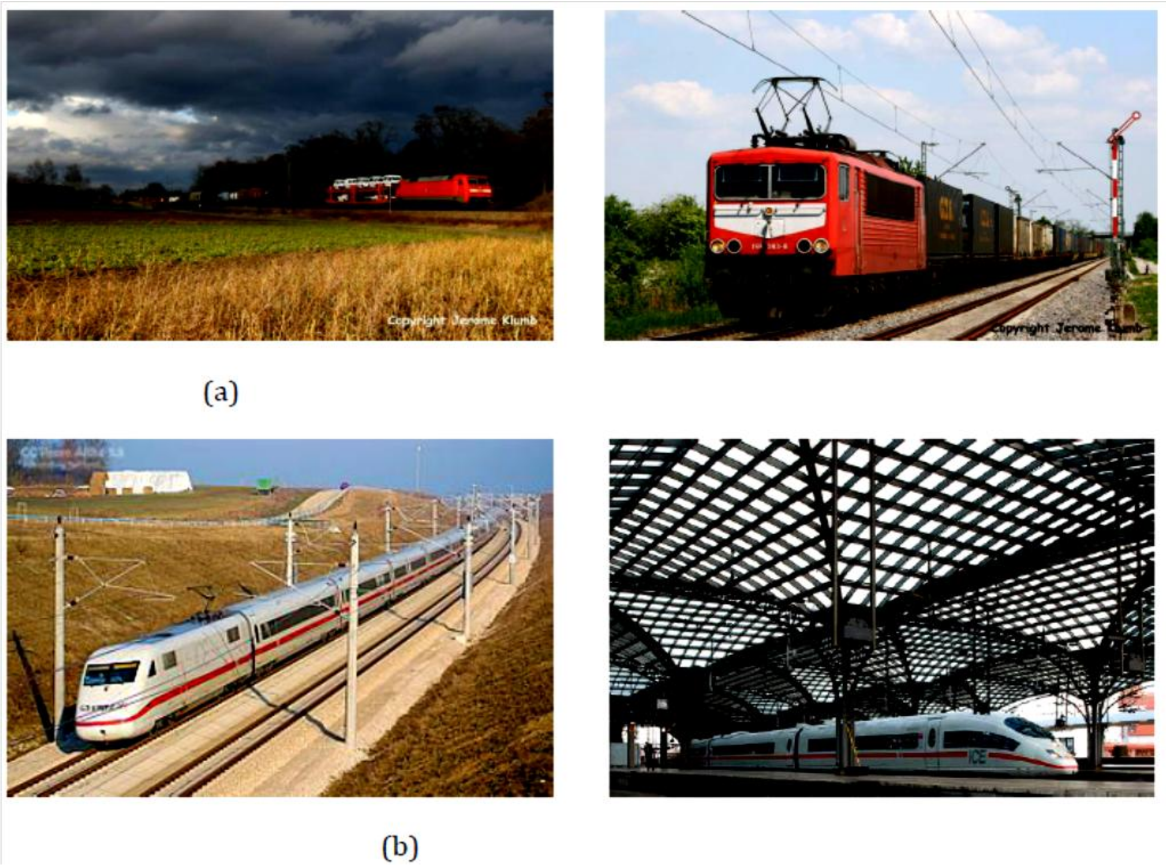


Figure 2.2: World's recent (a) cargo and (b) high speed trains with advanced infrastructures [Klumb, 2008; Siissalo et al., 2011]

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 [Coulls, 1999]. 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 [Coulls, 1999].

- 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.

2.2.2. Railway History of Ethiopia

The idea of constructing a railway to link the Ethiopian capital with the coast appears to have been first conceived by Menelik's Swiss adviser, Alfred Ilg. After countless negotiations with French, the concession was reached on March 9, 1894 and the plan was to build the railway from coast, Djibouti, to Entoto via Harar and then to the White Nile via Kaffa [Pankhurst, 2006].

The Swiss and French Engineer jointly formed a company in Ethiopia called *Compagnie Imperiale des Chemins de Fer Ethiopian* (Imperial Ethiopian Railway) that became operational in 1917 under a new name and status: *Chemin de Fer Franco–Ethiopien* (Ethio-French Railway) [Pankhurst, 2006; AtnafSeged Kifle et al, 2000].The first trains arrived at the capital, Addis Ababa, in 1917 by which date the service operated along a line of 784 kilometers.

The challenges in the progress of early railway developments were [Pankhurst, 2006]:

- Strong Ethiopian opposition of fearing in loosing territory on family of the King
- French need of colonial interest and hope of taking monopolistic position
- The financial difficulties and finally bankruptcy (in 1909) of the former firm

Figure 2.3: Ethio-Djibouti railway line [Ntamutumba, 2010]

The Ethio-Djibouti railway (CDE), like so many rail systems, was neglected for years in favour of road transport, but the loss of its main ports when Eritrea gained independence left Ethiopia totally dependent on Djibouti for an outlet to the sea. Some stretches of track are more than a century old; crumbling embankments and decaying bridges limit the weight and speed of the trains. A major restoration project is under way with European Union support, using heavier weight rails - 40kg per meter instead of the 20kg rails still in use on some stretches of the line [Blunt, 2009].

Now Ethiopia has launched the construction of a 5,000 km railway network which aims to link the capital, Addis Ababa, to various regions of the country which is part of the country's five-year transformation plan [ERC, 2011]. See Appendix C, Figure C-2 of Ethiopian national railway network map.

2.3. Track Geometry of Railway

2.3.1. Introduction

The location process for track geometry begins by roughly defining potential routes or areas through which a railroad might practically run. Additional and more detailed information is then collected, and the route alternatives are gradually reduced until the final route is chosen [Darvishsefat et al, 2004]. 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. 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 superelevation (transverse vertical plane) [Sadeghi, 2010].

2.3.2. Track Geometry Elements

Sensitive geometric elements of railway consist of alignment, transition length, track gauge and track cant or superelevation [Lindahl, 2001; Boraas, 2004].

2.3.2.1. 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 [Esveld, 2001]. As indicated in Table 2.1, various rail gauges have been adopted by different railways in the world, due to historical and other considerations [Handan, 2009].

Table 2.1: Various rail gauges on world railways [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

Even if there are various rail gauges, the standard track gauge is selected for this work. 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 (Esveld, 2001).

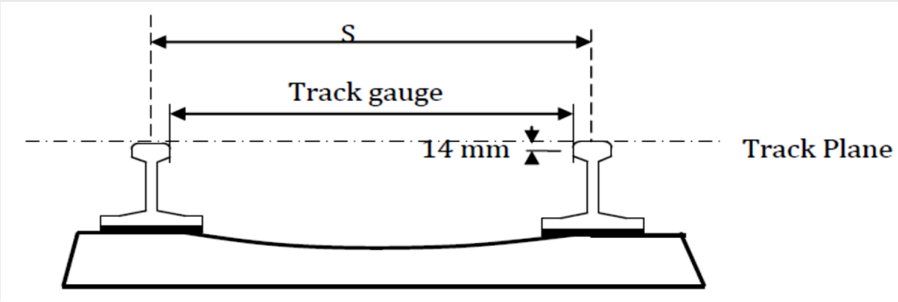


Figure 2.4: Track gauge (Lindahl, 2001)

Where: S is the center to center length of track gauge

2.3.2.2. Alignment

Boraas [2004] defined alignment as the route upon which a train travels and the track is constructed; and alignment can be defined in two fashions, horizontal and vertical alignment. First, the horizontal alignment defines physically where the route or track goes on (mathematically it is the XY plane). The second component is a vertical alignment, which defines the elevation, rise and fall (the Z component). The track alignment shall be designed to maximize passenger ride quality at the highest permissible operating speeds.

Horizontal curve

The most distinguished parameter for a circular curve is the radius, R constant and is related to the centre of track, which is inversely proportional to curvature, k [Esveld, 2001].

$$k = 1/R \qquad (2-1)$$

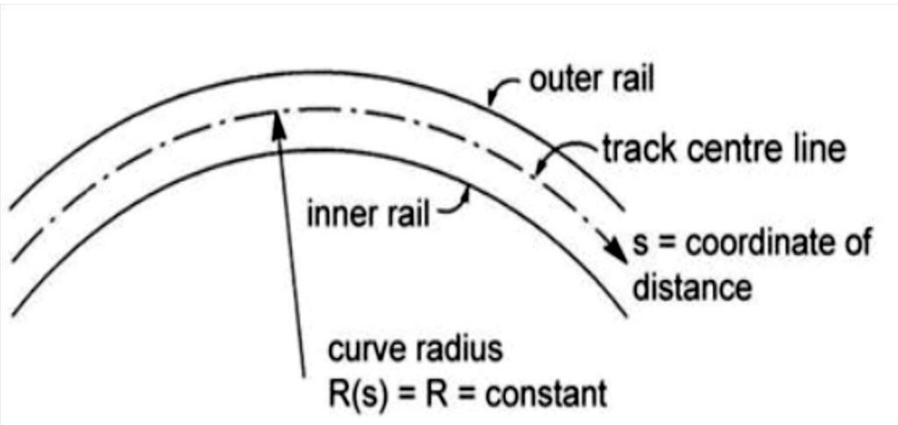


Figure 2.5: The definition of horizontal circular curve radius R [Lindahl, 2001]

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].

$$\alpha = \frac{1746}{R} \qquad (2-2)$$

It is a known fact that a vehicle running at a speed, v in a curve with a radius R undergoes a centrifugal lateral acceleration:

$$a_l = \frac{v^2}{R} \qquad (2-3)$$

High results of this acceleration in number cause undesirable effects such as [Esveld, 2001]:

- Possible passenger discomfort
- Possible displacement of wagon loads
- Risks of vehicle overturning in combination with strong side winds
- Risk of derailment caused by flange climbing of a wheel on the outer rail or by loosening of rail fastenings
- High lateral forces on the track

Gradient and Vertical Curve

The topographical conditions usually require some kind of vertical-longitudinal gradients, along the way. In particular heavy railway traffic has problems to overcome large longitudinal gradients. Therefore restrictions for the amount of gradient are needed. Large gradient results, principally, in heavier locomotives, increased locomotive power, and/or less freight train weight, reduced speed and line capacity, requirement of higher braking capacity for high-speed and freight trains, and/or larger signaling distances.

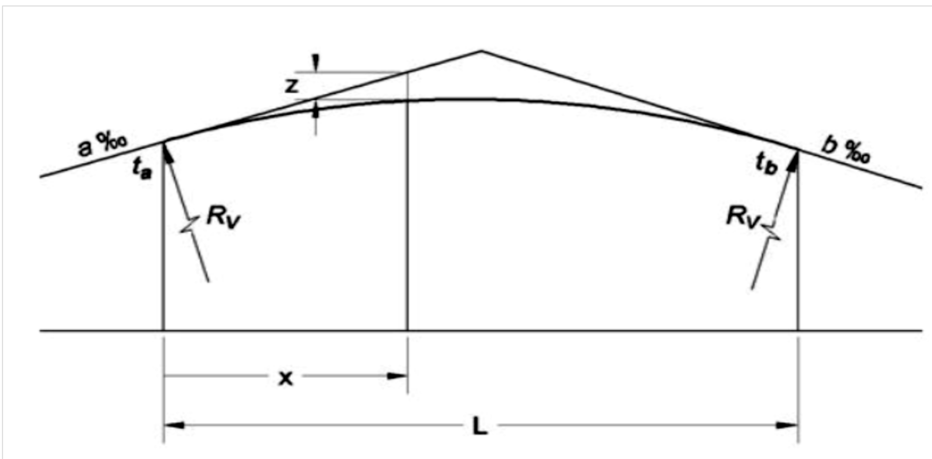


Figure 2.6: Vertical curve between two adjacent gradients

Where: L = length of vertical curve; a and b = gradients; R_v = Radius of vertical curve

Design of track gradients shall take account of the following factors [GC/RT5021, 2009]:

- Braking and traction performance of vehicles likely to use the line;
- 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);
- Projected rail adhesion conditions, including the effect of the weather; and
- The combined effect of gradient and horizontal curvature where the gradient coincides with a small radius horizontal curve.

A vertical curve, with suitable radius, provides a smooth transition between successive tangent gradients in railway profile [Lindahl, 2001]. In addition at changes in grade on main line, a vertical curve of sufficient length should be provided to prevent excessive slack action in long freight trains or a sensation of discomfort to passengers at maximum speed [Boras, 2004; GC/RT5021, 2009].

Transition curve

Transition curves are used between tangent track and circular curves or between two adjacent curves to allow a gradual change in curvature and lateral acceleration [Esveld, 2001]. Transition curves also introduce cant by means of transition gradients.

Transition curves are not used if

- The curve radius is > 3000 m;
- A calculation shows that no cant is necessary;
- Between two adjacent curves in the same direction the discontinuity in acceleration remains limited to 0.2-0.3 m/s^2.

Curvature shall increase (or decrease) regularly over the whole length of the transition curve. In addition, the clothoids spiral (or its close approximation, the cubic parabola) is the usual form of transition used on network rail controlled infrastructure.

Track Cant

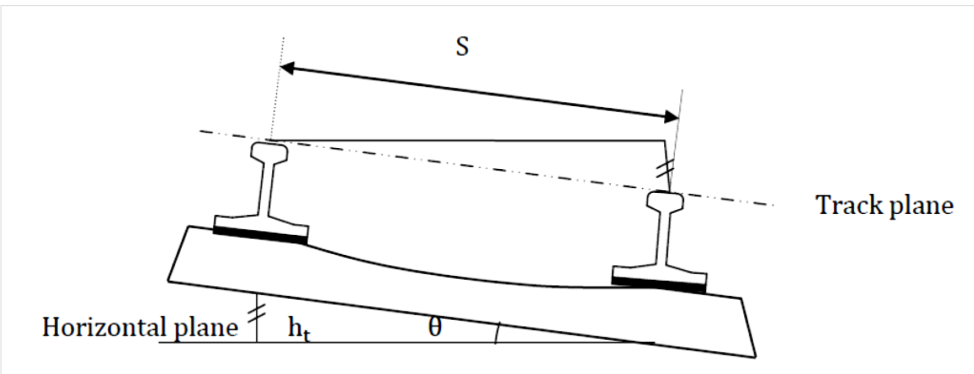


Figure 2.7: Track cant (Lindahl, 2001)

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

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 [Esveld, 2001; Lindahl, 2001].

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. Transition curves are used to introduce cant via superelevation ramps which is a section of the track where the cant changes gradually. Cant is normally applied to the high rail but may be split between high and low rails (if the low rail can be lowered).

$$\sin\theta = \frac{h_t}{S} \qquad (2-4)$$

According to Boras (2004) 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

2.3.3. Track -Vehicle Interaction

Lateral Acceleration

In case of quasi-static curving (i.e. curving at constant speed, radius and cant on perfect track geometry) the vehicle is exposed to two accelerations: horizontal centrifugal acceleration (parallel to horizontal plane) and gravitational acceleration.

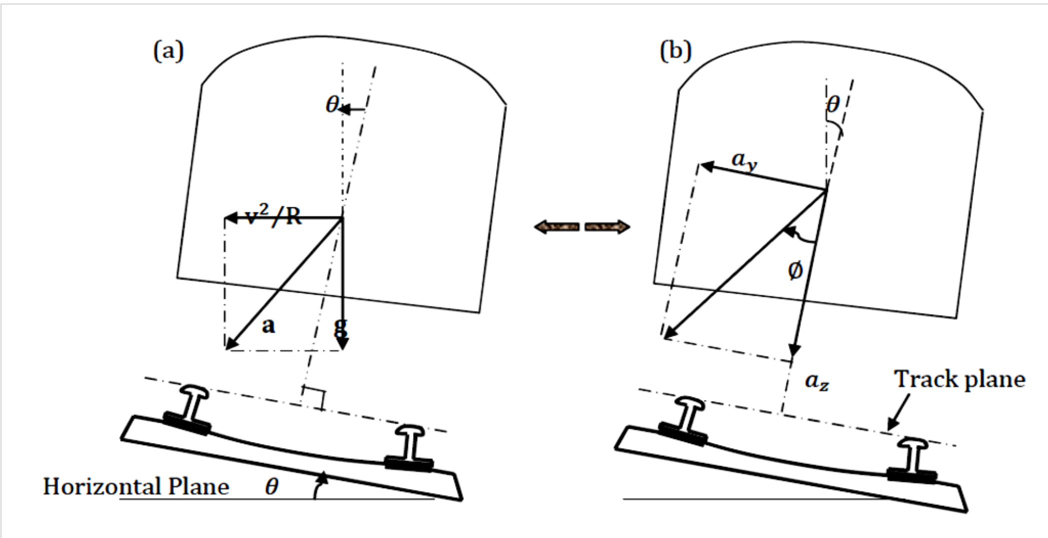


Figure 2.8: Rear view of rail vehicle car body at curving, (a) lateral acceleration in horizontal plane, (b) lateral acceleration in track plane [Thomas, 2009]

Where: $\theta =$ cant angle, $\emptyset =$ lateral force angle, g = gravitational acceleration, $v^2/R =$ centrifugal acceleration, a = resultant acceleration, a_y and a_z are component of resultant acceleration parallel and perpendicular to the track plane respectively.

Track plane acceleration or lateral acceleration, a_y

$$a_y = \frac{v^2}{R}\cos\theta - g\sin\theta = \frac{v^2}{R}\cos\theta - g * \frac{h_t}{S} \qquad (2-5)$$

$$a_z = \frac{v^2}{R}\sin\theta + g\cos\theta \qquad (2-6)$$

For small angle θ, $\cos\theta \approx 1$ and $\sin\theta \approx 0$

$$a_y = \frac{v^2}{R} - g * \frac{h_t}{S} \qquad (2-7)$$

$$h_t = \frac{S}{g}\left[\frac{v^2}{R} - a_y\right] \qquad (2-8)$$

$$a_z = g \qquad (2-9)$$

The lateral force angle, $\emptyset$ $\quad \emptyset = \tan^{-1}\left[\frac{a_y}{a_z}\right] \qquad (2-10)$

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. From equation (2-8) h_t becomes h_{eq} and rearranged as;

$$h_{eq} = \frac{S}{g} * \frac{v^2}{R} \qquad (2-11)$$

For the standard track gauge, S = 1500 mm, gravitational acceleration, g = 9.81 m/s^2, radius in m and speed V (km/hr); therefore, the equilibrium cant in mm

$$h_{eq} = \frac{1500\ \text{mm}}{9.81\ \text{m/s}^2} * \frac{V^2}{3.6^2 * R} = 11.8 * \frac{V^2}{R} \qquad (2-12)$$

The vehicle speed on a curve at which the resultant of the weight and the centrifugal force is perpendicular to the plane of the track or giving $a_y = 0$ for a given radius and cant is called the equilibrium or balanced speed, V_{eq} (AREMA, 2009 and Lindahl, 2001). Therefore, the components of the centrifugal force and the weight in the plane of the track are balanced.

For the cant in [mm] and radius in [m], the simplified form of V_{eq} in [km/hr] for the standard gauge is;

$$V_{eq} = \sqrt{\frac{R * h_t}{11.8}} \qquad (2-13)$$

Cant deficiency

Generally speaking however 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 [Esveld, 2001; Boras, 2004]. Cant deficiency h_d is the difference between ideal cant (equilibrium cant) and actual cant and must satisfy the condition;

$$h_d = h_{eq} - h_t \qquad (2-14)$$

Substitute equation (2-8) and (2-11) in equation (2-14) gives, when $a_y > 0$;

$$h_d = \frac{S}{g} * \frac{v^2}{R} - \frac{S}{g}\left[\frac{v^2}{R} - a_y\right] = \frac{S}{g} * a_y \qquad (2-15)$$

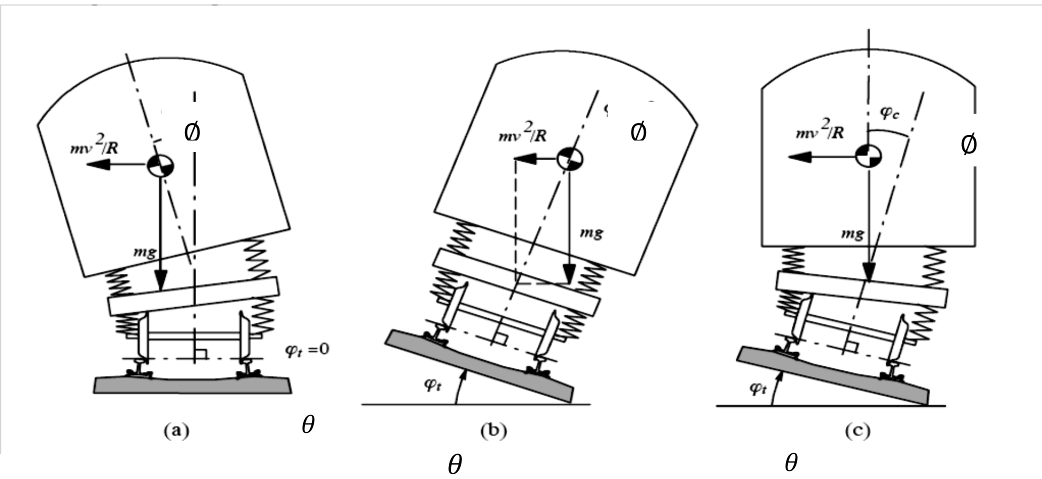


Figure 2.9: Rear view of rail vehicle at right-hand curving, (a) no cant but cant deficiency, (b) equilibrium cant, (c) cant and cant deficiency [Thomas, 2009]

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:

$$h_e = h_t - h_{eq} \qquad (2-16)$$

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. [Esveld, 2001; Lindahl, 2001].Cant excess is related with lateral acceleration in the same way of cant deficiency.

$$h_e = -\frac{S}{g} * a_y \qquad (2-17)$$

When $a_y < 0$, and $h_e > 0$.

Rate of cant and cant deficiency

With linear superelevation ramps according to Lindahl (2001), the rate of cant as a function of time can be expressed as:

$$\frac{dh_t}{dt} = \frac{v_{max} * \Delta h_t}{L_t} \qquad (2-18)$$

Where: Δh_t is cant variation, L_t is transition length and the units are in SI unit.

Similarly the rate of cant deficiency in SI unit can be described as:

$$\frac{dh_d}{dt} = \frac{v_{max} * \Delta h_d}{L_t} \qquad (2-19)$$

Relation between transition curve and superelevation ramp

According to Esveld (2001), transition curves and superelevation ramps should be arranged with linear curvature change (clothoids) and with linear changes of cant respectively.

$$kx = \frac{1}{r} \qquad (2-20)$$

$$h_x = \frac{h_t}{L_t}.x \qquad (2-21)$$

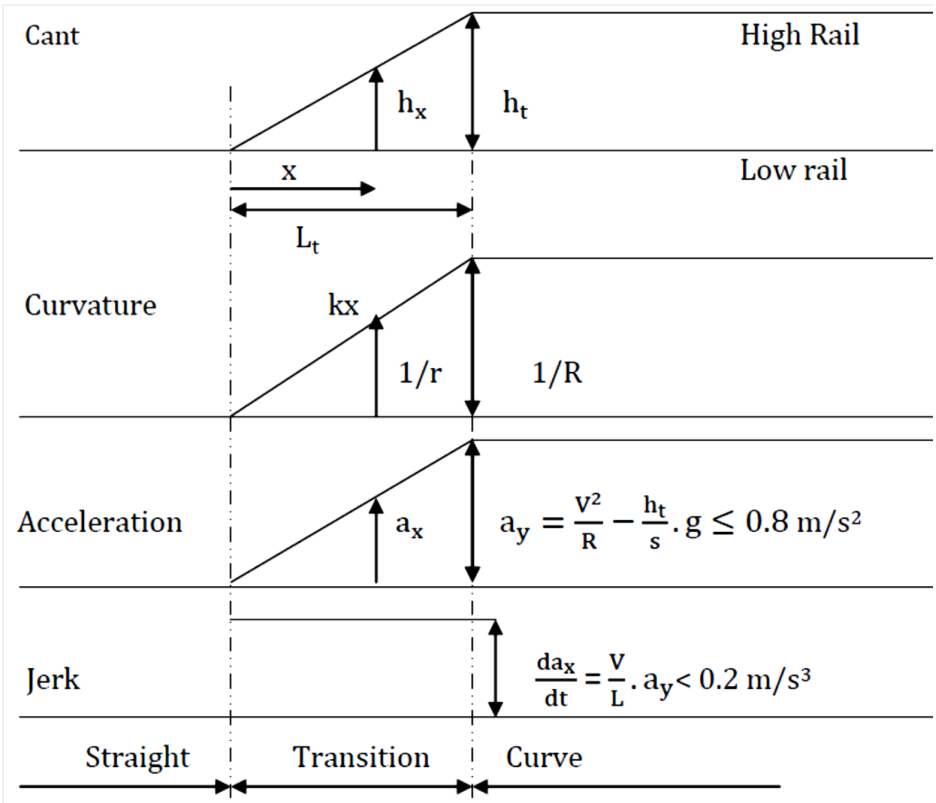


Figure 2.10: Relation between transition curve and superelevation ramp [Esveld, 2001]

Where: 1/r is the curvature of the transition curve that gradually increase from zero in the straight track to the value 1/R in the curve; 'k' is proportionality factor; h_x cant at distance 'x'; and 'x' is distance from the origin [Esveld, 2001 and Lindahl, 2001].

2.3.4. Track Geometry Standards and Practices

The standards and practices of Sweden, Germany, France, USA (AREMA), Technical Specification of Interoperability (TSI), and Railway Group Standards (RGS) on track geometry elements are being discussed.

The Railway Group Standards-GC/RT5021 (2009) set out requirements of track geometry that are applicable for speed less than or equal to 140 mph (225 km/hr) and axle loads not greater than 25.5 tones.

2.3.4.1. Track cant

The amount of cant is usually limited to 7 in (180 mm) to prevent undue tilting of the train if it stops on the curve [Boras, 2004].

Table 2.2: Limiting value of track cant [Lindahl, 2001; AREMA, 2009; TSI, 2008 and GC/GT5021, 2009]

Standard	Recommended Limiting Value (mm)	Exceptional Permissible Limiting Value (mm)
Sweden	150	-
Germany	100	180
France	160	180
TSI	160	180
CEN	160	160
AREMA	100	130
RGS	150	180

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

$$h_{t,max} \leq (R - 50) / 1.5 \qquad (2-22)$$

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

2.3.4.2. Track Cant Deficiency

For conventional trains on lines specially built for high-speed a limit value of 100 mm is recommended. However, in cases of very strict topological constraints a limit value of 130 mm is allowed.

Table 2.3: Track cant deficiency value [TSI, 2008; AREMA, 2009; Lindahl, 2001 and GC/RT5021, 2009]

Standard			Recommended Value (mm)	Max. Limiting Value (mm)	Permission (Exception) mm	Lateral acceleration a_y (m/s^2)
France			150	-	160	1-1.5
Germany			70	130	150	-
Sweden	Train-A		100	-	-	0.65
	Train-B		150	-	-	0.98
AREMA	Passenger		80	-	100- 130	-
	Freight		40	50	-	-
TSI	Passenger		150	-	-	-
	Freight		110	-	-	-
RGS			110	-	150	-
CEN	V≤ 160	Passenger	160	180	-	-
		Freight	110	160	-	-
	160<V≤200		140	160	-	-
	200<V≤230		120	160	-	-

Note:

- Category Train- A- conventional vehicles with older running gear and freight trains
- Category Train-B- vehicles with improved running gear according to approval
- Dashed for lateral acceleration indicates no data and others are not provided.
- The normal limiting design values for cant deficiency is at permissible speed for all standards on plain line CWR.

2.3.4.3. Track Cant Excess

The recommended and exceptional limiting design value for negative cant (cant excess) according to different standards are summarised as table below.

The earlier requirements on (a low) cant excess for slowly running freight trains had typical limit values of just 50 - 70 mm.

Table 2.4: Track Cant Excess value (Lindahl, 2001; TSI, 2008; GC/RT5021, 2009)

Standard		Recommended Limiting (mm)	Maximum Limiting (Exceptional) (mm)
France		70-100	105-135 (no freight)
Germany		100	-
CEN		110	130
RGS		80	-
Sweden	R > 1000	100	-
	R <1000	70	-

2.3.4.4. 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.

Using equation (2-12) - $h_{eq} = 11.8 * \frac{V^2}{R}$ and equation (2-14) - $h_{eq} = h_d + h_t$; the minimum allowable curve radius for the maximum and minimum operating speed shall be calculated using the following equations according to **CEN** provisional standard:

$$R = 11.8 * \frac{V_{max}{}^2}{h_t + h_d} \qquad (2-23)$$

$$R = 11.8 * \frac{V_{min}{}^2}{h_t - h_e} \qquad (2-24)$$

The minimum curve radius can be optimized so that the value of cant, cant deficiency and cant excess comply with the limits specified CEN provisional standard and satisfy the condition:

$$11.8 * \frac{V_{min}{}^2}{h_t - h_e} \geq R \geq 11.8 * \frac{V_{max}{}^2}{h_t + h_d} \qquad (2-25)$$

Where: V_i is in [km/hr], h_{eq} in [mm] and R_j in [m].

The Germany railway track horizontal curve radius is recommended based on an equilibrium cant of 170 mm, i.e. 100 mm of cant and 70 mm of cant deficiency; and the limit of horizontal curve radius (without permission) can be described based on an equilibrium cant of 290 mm using equation (2-23). On the other hand the recommended Swedish railway track horizontal curve radius, according to Lindahl (2001) review, is a value calculated from equation (2-23) with cant $h_t = 150$ mm and cant deficiency $h_d = 100$ mm in the formula for equilibrium cant, i.e. train category 'A'. For new lines they also recommended that the dimensional speed is multiplied by $\gamma = 1.3$ to get a margin with respect to ride comfort and increased speed in the future even if in reality it is often difficult to meet these recommendations.

According to RGS-GC/RT5021 (2009), the minimum horizontal curve radii shall be selected to take account of the curving characteristics of vehicles likely to use the track.

- The normal minimum radius shall be 200 m and 150 m on passenger and non-passenger running lines respectively.
- The exceptional minimum radius shall be 150 m and 125m on passenger and non-passenger running lines respectively.

A length of straight track not less than 3 m long shall be provided between the reverse curves if one of the curves has a radius of less than 160 m.

2.3.4.5. Transition curve and Superelevation ramp

The length of transition curve of Sweden railway according to Lindahl (2001) review is depending on the permitted gradient of cant and amount of jerk. The Banverket, Swedish, maximum rate of cant and rate of cant deficiency is shown in the table:

Table 2.5: Maximum rate of cant and cant deficiency and constants [Lindahl, 2001]

Train Category	Max. rate of cant [mm/s]	Max. rate of cant deficiency [mm/s]	q_t	q_d
A	46	46	6	6
B	55	55	5	5

Where: q_t and q_d are constants

The cant gradient in terms of change of cant and transition length, 1: n can be expressed as:

$$\frac{1}{n} = \frac{\Delta h_t}{1000 * L_t} \qquad (2-26)$$

The recommended length of transition curve according to Banverket [Lindahl, 2001] is:

$$\text{For } R \leq R_{rec} \qquad L_t = 5 * \sqrt{R} \qquad (2-27)$$

$$\text{For } R \geq R_{rec} \qquad L_t = \frac{V_{dim}^3}{9*R} \qquad (2-28)$$

$$L_t \geq 0.4 * \Delta h_t \qquad (2-29)$$

In addition the permitted speed V_{dim} [km/hr] in transition curve can be expressed as:

$$V_{dim} \leq \frac{1000 * L_t}{q_t * \Delta h_t} \qquad (2-30)$$

$$V_{dim} \leq \frac{1000 * L_t}{q_d * \Delta h_d} \qquad (2-31)$$

In Germany the maximum speed in transition curves for non-tilting trains, partly different from Sweden, can be expressed as (Lindahl, 2001):

$$V_{dim} \leq \frac{1000 * L_t}{8 * \Delta h_t} \qquad (2-32)$$

$$V_{dim} \leq \frac{1000 * L_t}{4 * \Delta h_d} \qquad (2-33)$$

The lower permissible limit of L_t according to this formula, equation (2-30), is applied on low speed track only; for high-speed lines the transition length is determined by the rate of change in cant deficiency.

The limiting values of the rate of cant as a function of length $(dh_t/dx)_{lim}$ shall apply to the following values, although not critical at high speed operation:

- Recommended limiting value: 2.25mm/m or 1:445
- Maximum limiting value: 2.5 mm/m or 1:400

The rate of cant and rate of cant deficiency should be less than or equal to their limiting values respectively.

$$\frac{dh_t}{dt} = \frac{v_{max} * \Delta h_t}{L_t} \leq \left(\frac{dh_t}{dt}\right)_{lim} \qquad (2-34)$$

$$\frac{dh_d}{dt} = \frac{v_{max} * \Delta h_d}{L_t} \leq \left(\frac{dh_d}{dt}\right)_{lim} \qquad (2-35)$$

The Recommended and maximum limiting values of the rate of cant deficiency as a function of time is 50 mm/s and 90 mm/s respectively for mixed traffic lines with passenger trains of speed, V≤230 km/hr. For all type of lines, the recommended and maximum rate of cant is 50 mm/s and 60 mm/s respectively. Therefore, the length of the transition curve L_t, the larger value can be selected from the following formula for selected value of dh_d/dt and dh_t/dx:

$$L_t \geq \frac{V_{max}}{3.6} * \Delta h_d \left(\frac{dh_d}{dt}\right)_{lim}^{-1} \qquad (2-36)$$

$$L_t \geq \Delta h_t \left(\frac{dh_t}{dx}\right)_{lim}^{-1} \qquad (2-37)$$

The normal limiting design value at permissible speed for rate of change of cant according to RGS-GC/RT5021 (2009) shall be 55 mm/s with exception of 85 mm/s. The normal limiting design values for rate of change of cant deficiency shall be 55 mm/s on plain line with exception of 70 mm/s. The steepest permitted designed cant gradient shall be 1 in 400.

2.3.4.6. Gradient

Quite light passenger trains with high traction forces and power (per ton of train) are able to climb much steeper grades than locomotive-hauled heavy freight trains. According to Lindahl (2001) review, Sweden railway allows a largest gradient of 1.0% on track with heavy freight trains and 1.25% if the mean value does not exceed 1.0% over each kilometer.

The Germany railway largest permissible gradient is 1.25% for mixed traffic main lines and 4.0% for commuter lines and secondary lines. Also, in new-build high-speed lines the higher gradient is used. According to TSI (2008) for high-speed lines dedicated to passenger traffic, gradients as steep as 3.5% shall be allowed for main tracks at the design phase; provided the following requirements are met:

- The maximum length of continuous 3.5% gradient does not exceed 6 km;
- The slope of the sliding average profile over 10 km is less than or equal to 2.5%.

In addition maximum gradients as steep as 1.25% is permitted for main tracks of mixed and freight lines. For sections up to 3 km the maximum gradient of 2.0% is permitted. For sections up to 0.5 km the maximum gradient of 3.5% is permitted in locations, where trains are not intended to stop and start in normal operation.

2.3.4.7. Vertical Curve radius

According to Banverket as reviewed by Lindahl (2001), the Sweden railway allowance of minimum and recommended vertical curve in accordance to permissible speed shall be expressed respectively as:

$$R_{v,min} \geq \frac{V_{dim}^2}{6.25} = 0.16 * V_{dim}^2 \qquad (2-38)$$

$$R_{v,rec,min} \geq 0.25 * (1.3 * V_{dim})^2 \qquad (2-39)$$

Table 2.6: Design value for vertical curve radius for high speed lines of Germany railway (Lindahl, 2001)

		Minimum vertical Curve Radius $R_{v,min}$
Without permission	Recommended Value	$0.4 * V_{dim}^2$
	Limit Value	$0.25 * V_{dim}^2$
Permission Necessary	Permission	$0.16 * V_{dim}^2$ on a crest
		$0.13 * V_{dim}^2$ in a hallow
		≥2000 m

According to TSI cited by Lindahl (2001) the design vertical curve shall be:

$$R_v = \frac{V_{max}^2}{12.96 * a_v} \geq R_{v,lim} \qquad (2-40)$$

Where: 12.96 is conversion factor of speed from km/hr to m/s ($3.6^2 = 12.96$)

The vertical acceleration a_v, shall be selected taking into consideration of ride comfort where there is a possibility of a non-optimal track bed. Human reaction to vertical acceleration magnitude is tabulated in Appendix C of Table C-1. The recommended and maximum limiting values of vertical acceleration, a_v for mixed traffic line with passenger trains of speed less than or equal to 230 km/hr are 0.22 m/s^2 and 0.31 m/s^2 respectively.

The normal limiting design value for vertical curve radii according to RGS and TSI shall be 1000 m. The maximum vertical acceleration, according to TSI, experienced in a vehicle due to the effect of the vertical curve shall be 0.06 g.

The minimum length of the vertical curve for both sags and summits, according to AREMA (2009), is determined by the following formula (except that in no case should the length of the vertical curve be less than 100 feet long):

$$L_v = \frac{A * V_{max}^2}{12.96 * a_v} \qquad (2-41)$$

Where: L_v is length of vertical curve, A is Absolute value of the difference in rates of grades expressed as a decimal and a_v is vertical acceleration.

2.4. Railway Track Structure

2.4.1. Introduction

The purpose of a railway track structure is to provide safe and comfortable train transportation. This requires the track to serve as a stable guide-way with appropriate vertical and horizontal alignment. To achieve this role, each component of the system must perform its specific functions satisfactorily in response to the traffic loads and environmental factors imposed on the system [Ionescu, 2004].

The main types of rail track for urban and regional rail applications, according to Heunis (2011), are: ballast, covered and slab track.

A. Ballast track: are mainly used for regional transport of passengers and goods. It consists of rails and sleepers mounted on a ballast bed.

B. 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.

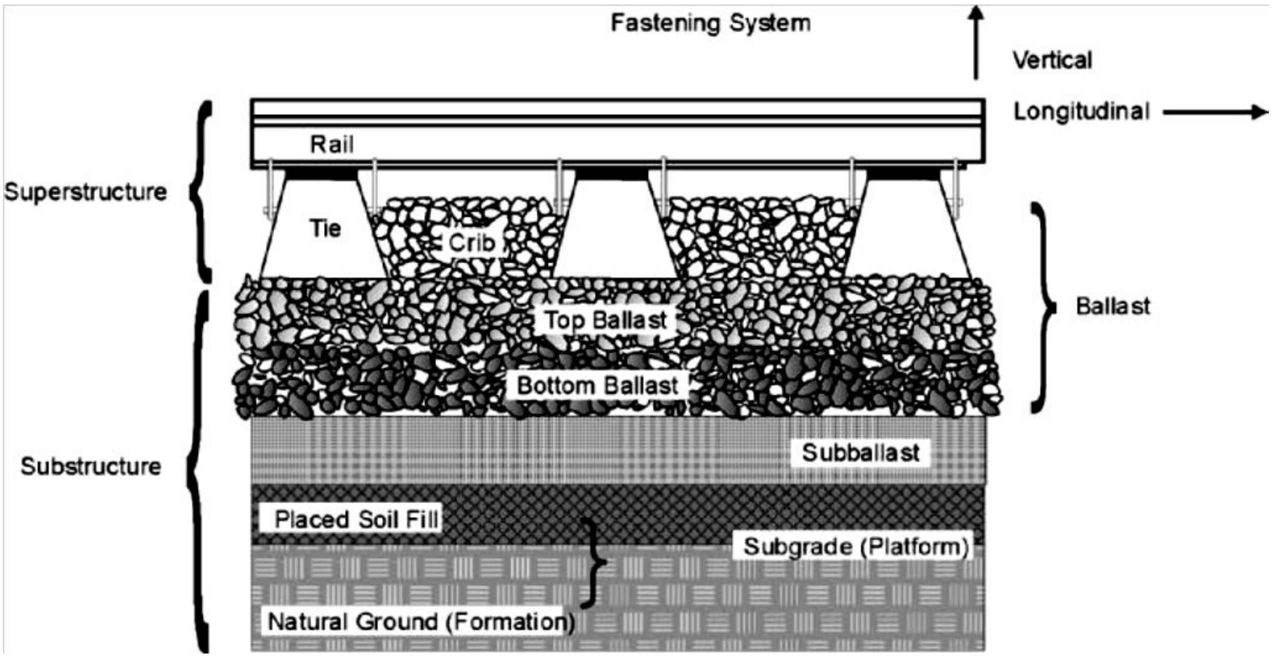
C. Slab track: are mainly used for high-speed rail tracks, tracks in tunnels, tracks on bridges and tracks which require little maintenance (e.g. covered tracks and green tracks). It consists of rails and/or sleepers mounted or cast into a solid base.

Table 2.7: Advantage and disadvantage of track type [Heunis, 2011]

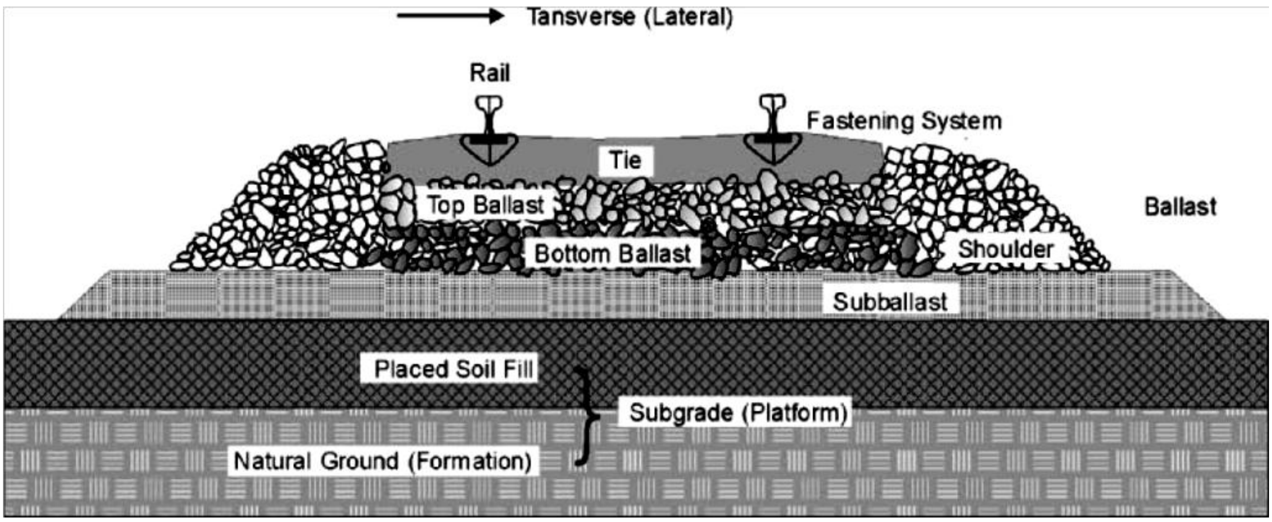
Track Type	Advantages	Disadvantages
Ballasted	Low construction costs and inherent vibration damping	Ballast may degrade and need replacement or realignment i.e. costly and time consuming
Covered	Track areas can be used by other modes of transport	Replacement is costly, timely and disruptive to other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

The component typical track structures, ballast track, are divided into two parts, namely the superstructure and substructure which are the top and bottom parts. The superstructure refers to the top part of the track includes the rails, fastening system, and sleepers; and conventional ballasted substructure is a layered system; its components include ballast, sub-ballast, and subgrade soil providing support to the rail and sleepers [Ionescu, 2004;

Aursudkij, 2007; Lim, 2004]. While the superstructure provides the main function of the railway, the substructure provides the foundation to support the superstructure and to help the superstructure to reach its optimum performance [Aursudkij, 2007; Berggren, 2009]. The ballast and subballast layer in granular trackbed can be replaced with a HMA layer in HMA trackbed for slab track [Rose and Konduri, 2006]. Capping layer can be provided to protect the natural ground or fill from moisture ingress and to form a unified subgrade layer [Radampola, 2006].



(a)



(b)

Figure 2.11: Components of track structure (a) longitudinal section; (b) transverse section (Selig, 2004)

2.4.2. Rail

The rails are a pair of longitudinal steel members which are in contact with the train wheels; and their primary functions are to guide the train in the desired direction and to transfer the traffic loading to the sleepers which are joined to the rails by the fastening system [Esveld, 2001; Selig, 2004; Lim, 2004; and Aursudkij, 2007].

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 [Pen, 2008].

2.4.2.1. 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. Typically rails experience excessive wear during service and often take place at rail joints or fishbolt holes of jointed track and in contact between the sides of the flange of a wheel and the gauge face of the rail. As a result the rail requires grinding to maintain a smooth running service at regular intervals and, when necessary, replacement is done or, if jointed rail, it is changed to continuously welded rails (CWR) in order to obtain extra life [Bonnett, 2005 and Pen, 2008].

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 [Boras, 2004]. The most commonly used rail profile, Appendix C of Table C-1, is flat-bottom rail, also called Vignole rail, and is divided into three parts: rail head, rail web and rail foot. The mass per metre of rail is known to contribute to the track stability and for high speed lines a mass of about 60 kg per metre (Specification: 60 E1) is the norm in Europe [Pen, 2008].

Steel Rail Grades

The standard steel rail grades could be designated as R200 (700), R260 (900A), R260 Mn (900 B) and R320 Cr (1100) and represented by the Brinell Hardness [Jain and Murthy, 2008].

Table 2.8: Prescribed chemical composition and tensile properties of a rail steels according to UIC 860 V8 [Vitez et al, 2004; Jain and Murthy, 2008]

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

Note: * - Current European rail grade properties are shown in Appendix-C of Table C-2.
** - Other alloy elements such as vanadium (V) or molybdenum (Mo), niobium (Nb) can be applied according to agreement between manufacturer and buyer.

Carbon (C) content in rail increases tensile strength whereas other alloy elements such as Manganese (Mn) make steel more resistance to abrasive forces, increase strength, toughness and elasticity; Silicon (Si) produces denser steel; and Cr, V, Mo and Nb are also effective alloys (Jain and Murthy, 2008).

Table 2.9: 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 (Xiao-fei et al, 2001).

2.4.2.2. Loading of Rail

The forces imposed on the track structure could be classified as mechanical (both static and dynamic) and thermal. Esveld (2001) discussed the type of forces and their source as: (a) quasi-static loads induced by the self-weight of the vehicle and reaction forces in curves; (b) dynamic loads resulting from track irregularities; and (c) thermal loading due to temperature variations in continuous welded rail (CWR).

Vertical Load: vertical forces are perpendicular to the plane of the rails and may be vertical wheel load or uplift force (reaction to wheel load) and are those forces result the mechanical stresses in the track [Selig and Waters, 1994 and Profillidis, 1995 cited by Ionescu, 2004]. The general method used in the determination of the design vertical wheel load according to Doyle (1980) is to empirically express it as a function of the static wheel load, i.e.

$$P = \emptyset P_s \qquad 2-42$$

Where: P=design wheel load (kN), P_s = static wheel load (kN), and Ø = dimensionless impact factor (always >l).

Dynamic Impact Factors

The nominal vehicle axle load is usually measured for the static condition, but in the design of railway track the actual stresses in the various components of the track structure and in the rolling stock must be determined from the dynamic vertical and lateral forces imposed by the design vehicle moving at designed speed [Doyle, 1980]. Dynamic impact factor is a corrective factor to compensate for dynamic as well as impact effects of wheel load resulted from wheel and rail surface irregularities [Sadeghi and Barati, 2010].

Table 2.10: Recommended relationship for dynamic coefficient factors [Doyle, 1980; Sadeghi and Barati, 2010]

Recommender	Formula
AREMA	$\emptyset = 1 + 5.21\frac{V}{D}$
DB	$\emptyset = 1 + \frac{V^2}{3*10^4}$ for V≤100 km/hr $\emptyset = 1 + \frac{4.5*V^2}{10^5} - \frac{1.5*V^3}{10^7}$, for $V > 100$km/hr
India	$\emptyset = 1 + \frac{V}{58.14k^{0.5}}$
South Africa	$\emptyset = 1 + 4.92\frac{V}{D}$ for narrow gage

Where: V= Vehicle speed (Km/hr); D= wheel diameter (mm); and k = track modulus (Mpa)

2.4.2.3. Rail Bending Stresses

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 [Krishna et al, 2006].

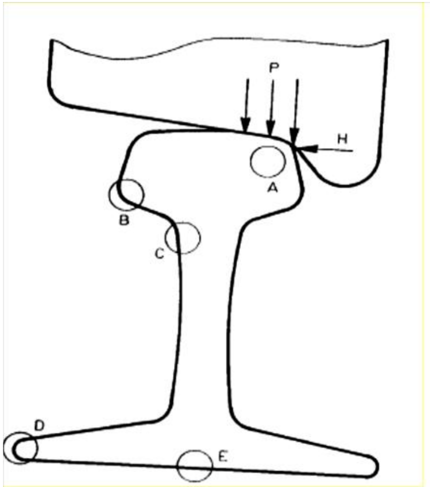


Figure 2.12: Loads on the rail and positions of high rail stresses [Krishna et. al., 2006]

Rail Bending Stresses in the Rail Base

Beam on elastic foundation theory is used to predict stresses in the rail. The relationship between rail bending moment, applied load, support conditions and location is given by:

$$M_x = \frac{P\ e^{-\beta x}}{4\beta}\ (\cos\beta x - \sin\beta x) \qquad 2-43$$

$$\beta = \sqrt[4]{\frac{k}{4EI}} = \text{stiffness ratio} \qquad 2-44$$

Where: - M_x is rail bending moment a distance "x" from the load source

- P is the dynamic single wheel load on the track (i.e. $P = \emptyset P_s$)
- k is the track modulus
- E is the modulus of elasticity of the rail
- I is the moment of inertia of the rail section
- x is the longitudinal distance along the track from load source

For design purposes, the maximum dynamic bending stress in the rail is [AREMA Vol.4 chap.16, 2009]:

$$\sigma_{max} = \frac{M_{max} * c}{I} = \frac{M_{max}}{Z_b} \qquad 2-45$$

Where: c = distance from neutral axis to rail base,

Z_b = section modulus for rail base and

M_{max} = dynamic maximum bending moment due to all wheel loads.

Allowable Rail Bending Stress

Under various operating conditions the allowable bending stress should be sufficiently below the elastic limit (or yield stress) of the rail steel to avoid fatigue cracking [AREMA, 2009 and Krishna, 2006].

Table 2.11: Allowable rail bending stress [AREMA, 2009; Krishna et al, 2006]

Approach	Allowable Rail bending stress (σ_{all})	Remark
AREMA	$\frac{\sigma_y - \sigma_t}{(1+A)(1+B)(1+C)(1+D)}$	σ_{all}= 25,000 psi Severe assumption A=20%, B=25%, C=15%, D=15% and σ_t =20,000psi
Germany	$1.6\ \sigma_b + \sigma_t \leq \sigma_{all} = 0.9\ \sigma_y$	$\sigma_b \leq 0.5625(\sigma_y - 1.111\sigma_t) = (\sigma_{all})'$
Australia	$0.4 \times f_{yk}$	Not consider the effect of corrosion and sharp surface
India	$0.29 \times f_{yk}$	LWR track

Where:- σ_b, σ_y and σ_t are calculated rail bending stress at the centre of the rail base, yield stress of the rail steel and temperature induced stress in the rail (longitudinal stress)

- A, B, c and D are stress safety factor in percent to account for lateral bending of the rail, track conditions, rail wear and corrosion and unbalanced superelevation of track.
- $\sigma_t = E * \alpha * \Delta t$; E - Elastic modulus of steel rail, $\alpha = 6.5 \times 10^{-6}$ - coefficient of thermal expansion (/°F or /°C) for steel and Δt is change in temperature (°C or °F)

Rail Shear Stress

The rail is analysed, after bending stress criteria of rail is satisfied, to establish its capacity to withstand the contact stresses at the point of wheel rail/interaction [Krishna, 2006]. The elastic deformation of the steel of the wheel and the rail creates an elliptic contact area and the mean contact stress can be expressed as [Krishna et.al, 2006]:

$$q_{mean} = \sqrt{\frac{E * Q}{64\ R_w b\ (1 - \upsilon^2)}} \qquad 2-46$$

Where: q_{mean} =Mean contact stress (N/m^2 or Pa)

E = Modulus of elasticity, 207Gpa (30×10^6Psi)

Q = P = effective wheel load (N)

R_w = wheel radius (m)

2b = width of wheel/rail contact area (m)

υ = poison's ratio (υ = 0.3 for steel rail)

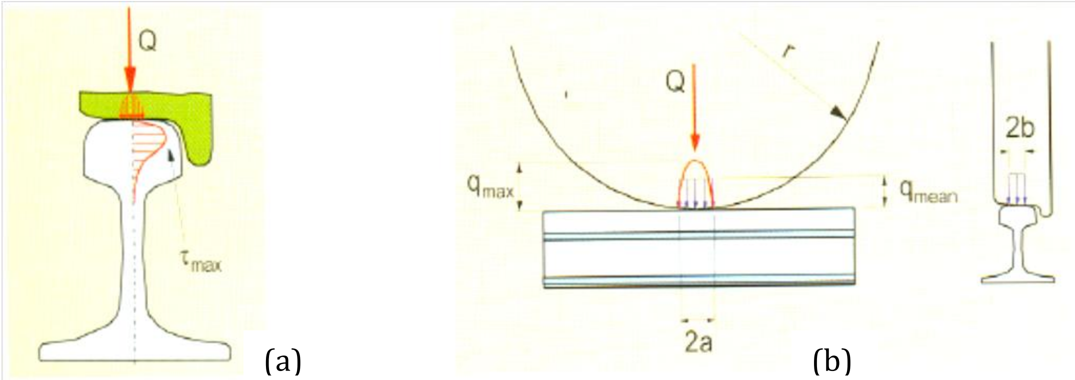


Figure 2.13: (a) Shear stress distribution in rail head and (b) Assumed contact distribution between wheel and rail [Krishna et.al, 2006]

Table 2.12: Maximum and permissible shear stress in the rail head [Krishna et.al, 2006]

Shear stress type	Formula	Remark
Maximum shear stress (τ_{max})	$0.3 * q_{mean}$	Occurs at a depth of 4-6 mm in the rail head
Permissible shear stress (τ_{all})	$f_{yk}/\sqrt{3}$	No account of fatigue nature of the load
Permissible shear stress (τ_{all})	$0.3f_{yk}$	Account of the fatigue nature of the load, 50% of tensile strength is considered

2.4.2.4. Selection of rail grade

The various criteria used to choose a steel grade are grouped under the following three headings (Vitez et al, 2004; Pointner, 2010).

Local Parameters: influence the development of wear (dominant at curve radius less than 1000m) and rolling contact fatigue (occurs over a radius range of 500 to 5000m) defects. The parameters are curve radius, amount of tonnage, gradients (greater than or equal to 2%) and speed and cant on curves axle load (the higher axle load is conducive to crushing on the low rail) and the type of rolling stock.

Rail Maintenance Methods: maintenance methods (lubrication and grinding) help to combat wear and RCF. Lubrication is a means of retarding rapid lateral wear of the high rail in curves whereas grinding prolongs the service life by preventing the emergence of defects.

Economic Issues: the economic consideration is the most relevant criteria. The increased investment cost of premium steel grades for rails will be justified by decreased cost figures for maintenance and operation by better performance, better safety and better availability. The result is a reduction of total life cycle cost (LCC).

The most appropriate steel rail grade should yield the lowest average annual overall cost. The average annual overall cost consists of the investment cost and maintenance cost and may be defined the following general formula (Vitez et al, 2004).

$$P = \frac{C}{n} + M \qquad (2-47)$$

Where: P – average annual overall cost, C – investment cost, n – service life of rails (in years), and M – annual maintenance cost

The present guidelines for rail grade selection on mixed traffic lines with up to 22.5 tonne axle load and at least 20 MGT annual loads show a wide variation between the different railways, especially for the shallow curves with radius less than 1500 m. While in Italy, Ireland and the UK, rail grade R260 is recommended for the whole range of curvature, several countries use R350HT (heat treated C-Mn [R260 + up to 0.15% Cr]) up to 500 m or 700 m wide curves to combat wear. DB increased their guidelines for the use of R350HT to 1500 m to combat RCF (Pointner et al, 2010). UIC Recommendation for the use of standard and hard steel grades for different horizontal curve radius is shown in Appendix-C of Figure C-4.

2.4.2.5. Rail Design and Specification

The goal of rail cross-section design should be to select the shape, size, material, and rail hardness to provide the most economical rail with the required strength and ductility for wear durability (Agarwal, 1998; Marich et. al, 1991 as cited by Selig (2004)).

Rails must have sufficient stiffness (EI) to distribute wheel loads over sleepers and limit deflection between the supports. Rail defects and discontinuities, such as joints, can cause large impact loads, which have detrimental effects on the track components below [Lim, 2004].

The rail section can be chosen using the design criteria of bending stresses of $\sigma_{max} \leq \sigma_{all}$, it follows that $M_{max}/Z_b \leq \sigma_{all}$. Thus, the required section modulus is [AREMA, Chap.16, 2009]:

$$Z_b \geq \frac{M_{max}}{\sigma_{all}} \qquad 2-48$$

The permissible wheel load or wheel radius can be estimated using the shear stresses criteria of $\tau_{max} = 0.3 * q_{mean} \leq 0.3f_{yk} = \tau_{all}$ [Krishna et. al, 2006]. Therefore, from equation (2-46)

$$q_{mean} = \sqrt{\frac{E * Q}{64\ R_w b\ (1 - \upsilon^2)}} \leq f_{yk} \qquad 2-49$$

$$P = Q \leq \frac{f_{yk}^2 * 64\ R_w b\ (1 - \upsilon^2)}{E} \qquad 2-50$$

2.4.3. 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 [Lim, 2004; Pen, 2008].

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 (Esveld, 2001; Selig, 2004 and Bonnett, 2005).

2.4.3.1. Types of sleeper Materials

The possible types of material for sleeper can be timber, steel, Concrete, and composite materials (Esveld, 2001; Selig, 2004; Boras, 2004; and Bonnett, 2005).

Timber Sleepers

Timber sleeper was accepted by most railways as standard up to about the middle of the twentieth century although its durability limitations were recognised. Even today there are still many railways using timber sleepers, where the advantages of good resilience, ease of handling, adaptability to nonstandard situations or electrical insulation are very important [Bonnett, 2005].

Steel Sleepers

Steel sleepers have been hardly ever used in the UK, largely because of cost and fear of corrosion in variable weather conditions and are also used in countries particularly where trains run at moderate speeds only [Bonnett, 2005]. Even if steel sleepers have a few strong points such as: long service life, great dimensional accuracy and positive residual value; Esveld (2001) states that they are now used on a very small scale because of problems such as: insulation, maintenance using tampers and relatively high price.

Concrete sleepers

The development and use of concrete sleepers became significant after the Second World War owing to the scarcity of wood, the introduction of continuous welded rail (CWR) track, and the improvements in concrete technology and pre-stressing techniques [Esveld, 2001].

Specific advantages of concrete sleepers according to Esveld (2001) and Selig (2004) are: producing stable connection of CWR track due to its heavy load (200-300 kg); encouraging great freedom of design and construction; relatively simple to manufacture; and used to provide a cant to the rails to help develop proper rail wheel contact. In addition according to Kaewunruen and Remennikov (2008) they are more widely used than others since they are not affected very much by either climate or weather.

On the other hand the drawbacks of concrete sleepers according to Esveld (2001) are: 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; and residual value is negative.

According to Esveld (2001) concrete sleepers can be either two-block or mono-block. Mono-block 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 2.14: Types of concrete sleepers [Kaewunruen and Remennikov, 2008]

Selecting Type of Sleeper Material

The parameters used to select the preferred crosstie material according to Keating et al (2002) include life-cycle costs, crosstie design methodology, and rail fastening systems.

The general comparison can be based on the economic analysis that consists of the cost per tie including fastening system and the number of ties per mile. If there is sufficient data to determine the average usable life for concrete ties, lifecycle costs can be included in this analysis (Keating et al, 2002). Generally lifecycle costs of concrete sleeper are assumed to outlast timber but be more expensive to replace.

2.4.3.2. Spacing and Dimensions of Sleepers

A. 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 can reduce the fatigue life of the rail even though the stress levels are within allowable limits [AREMA, 2009].

According to AREMA Sec. 4.3.2.1 (2009), the recommended practices of using center-to-center spacing's of ties between 20 inches (510 mm) and 30 inches (760 mm) intended for track designs. In Australia standards [AS1085.14, 2003], a centre to centre spacing's of 500 mm to 750 mm prestressed concrete sleepers are intended for track designs.

B. 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 [AS1085.14, 2003]. Sleepers shall be designed to a preferred length of 2.5 m and a maximum depth at the rail seat of 250 mm.

According to AREMA Sec. 4.3.2.1 (2009) the recommended practices cover tie designs 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. Because of bond transfer, pretensioned concrete ties shall be at least 8ft. 0 inches (244 cm) long unless additional provisions are made to ensure adequate bond transfer. In addition, the minimum and maximum design 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.

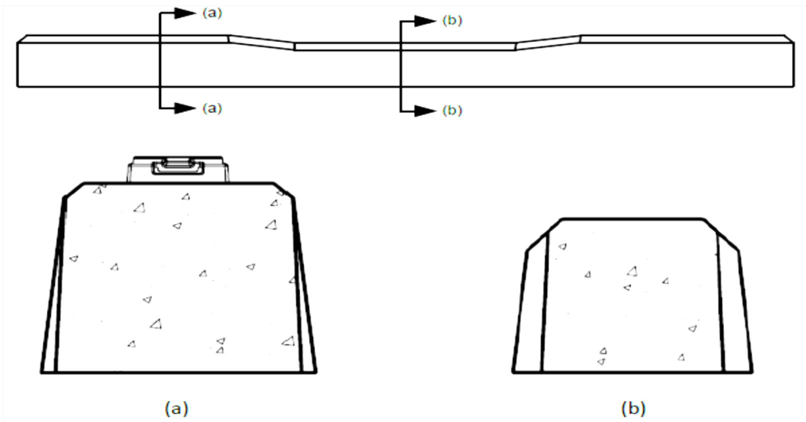


Figure 2.15: Typical tie cross section profile (a) rail seat section (b) center section [Lutch, 2009]

C. Cross-section of sleeper

Cross section of sleeper selection is the first step for numerical analysis of PSC design; using the parallel axis theorem, values for the moment of inertia of the sections were obtained both at the rail seat and center critical sections (Lutch, 2009).

2.4.3.3. Loading of Sleepers and stresses

A. Rail Seat Load

According to Krishna et al. (2006), the exact magnitude of the load applied to each rail seat depends upon the parameters such as: the rail weight, the sleeper spacing, the sleeper stiffness, track modulus, pad stiffness, and the amount of play between the rail and the sleeper. 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 and Barati, 2010].

Table 2.13: Comparison of the various methods used for calculation of the maximum rail seat load [Doyle, 1980; AS 1085.14, 2003; Sadeghi and Babaee, 2006]

Methods	Formula	Maximum rail-seat load
3 Adjacent sleepers method	$q_r = 0.5P$	$q_r = 0.5P$
Australia Standards (Prestressed Concrete sleepers at 750 mm centers)	$q_r = DF * P$ where $P = jP_s$	$q_r = 0.6P$*
AREMA method (Pre-stressed concrete sleepers at 760 mm centers)	$q_r = DF * P$	$q_r = 0.6P$*
Note: * The rail seat load is at maximum sleeper spacing and for given spacing it is determined from Appendix C, Figure C-3.		

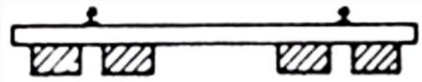
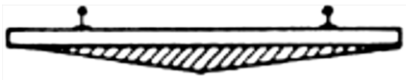
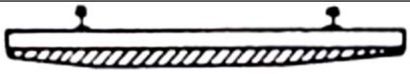
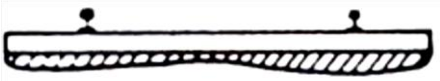
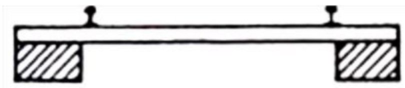
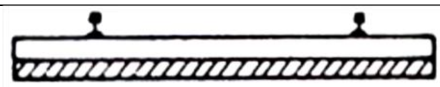
Where: P = design wheel load; j= factor for combined quasistatic and dynamic design load ($j \geq 2.5$, AS 1085.14, 2003); and DF= load distribution factor.

B. Ballast/Sleeper Contact Pressure

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; Sadeghi and Babaee, 2006].

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 [Selig, 2004]. In order to calculate contact pressure between the sleeper and ballast several approaches have been developed.

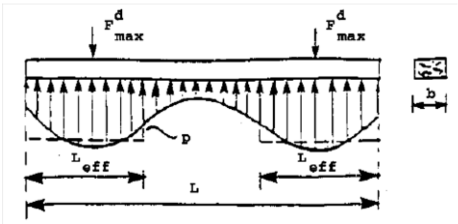
Table 2.14: Hypothetical distribution of sleeper bearing pressure (current practices) [Sadeghi and Babaee, 2006]

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

I. AREMA Method

While tie-to-ballast pressure is not uniformly distributed across or along the bottom of a cross tie, an approximate calculation can be made of "average" pressure at the bottom of the tie. The average intensity of pressure on ballast (P_a) should not exceed 65 psi (450 kPa).

Table 2.15: Maximum Ballast-Sleeper Pressure [AREMA, 2009]

	Effective Length of ballast support beneath each rail seat (L) (m)	Ballast-sleeper bearing pressure (P_a) (kPa)
	$L_{eff} \cong {}^{l}/_{3}$	$P_a = \frac{3 * q_r}{b \times l}$

Where: b = width of tie at base; l = length of sleeper; and $F^d_{max} = q_r = P_s . DF [1 + \emptyset]$- rail seat load, P_s = Wheel load in pounds (KN); $\emptyset$ = Impact factor in percent ($\emptyset = 2$ is assumed); DF = Distribution factor (from Appendix C, Figure C-4)

Adjustments of this limitation may be dictated by differences in the ballast abrasion resistance, the ties' flexibility, size and spacing. The recommended ballast pressure should not exceed 85 psi (586 KPa) for high-quality, abrasion resistant ballast [AREMA, 2009].

II. Australia Standards Method

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 2.16: Maximum ballast-sleeper pressure for standard gauge [AS 1085.14, 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	$P_a = \dfrac{q_r}{b \times (l-g)}$

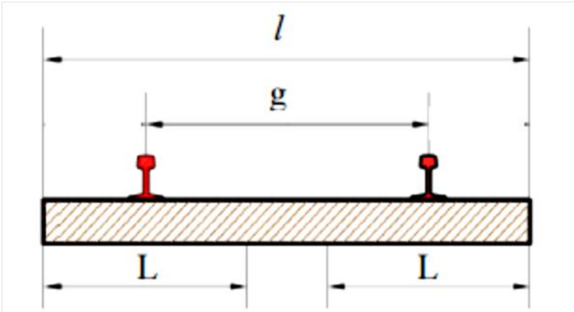


Figure 2.16: Effective length (area) of sleeper support at rail seat [Sadeghi and Barati, 2010]

Sleeper Bending Moment

The flexural capacity of a concrete tie is derived from material properties, tie dimensions and number and type of prestressing wire used (Lutch, 2009). The maximum bending moments are located at the rail seat and at the center of the tie length; according to Freudenstein (2007) crosstie must be capable of bearing this design moment without cracking.

Table 2.17: Different recommended methods for calculation of prestressed monoblock concrete sleeper bending moments [AS1085.14, 2003; AREMA Sec. 4.4.1.2, 2009]

Developer	Rail Seat Moment		Center Moment	
	M_r^+ (kN.m)	M_r^- (kN.m)	M_c^+ (kN.m)	M_c^- (kN.m)
AREMA	$M.V_f.T_f$	$f_1.M_r^+$	$f_3.M_r^+$	$f_2.M_r^+$
Australia Standards (Standard gauge)	$q_r\left(\dfrac{l-g}{8}\right)$	Max{$0.67M_r^+$,14}	$0.05q_r$ (l-g)	$q_r\left(\dfrac{2g-l}{4}\right)$

Where:

- M = unfactored bending moment in inch kips (kN-m) taken from Appendix C, Figure C-5
- V_f and T_f are speed and tonnage factor obtained from Appendix C, Figure C-5
- f_1, f_2, f_3 are factors depending on length of ties

Table 2.18: Values of moment factors for monoblock ties related to tie length [AREMA Sec. 4.4.1.2, 2009]

Tie Length	f_1	f_2	f_3
7'9" (2.360 m)	0.72	1.13	0.61
8'0" (2.440 m)	0.64	0.92	0.56
8'6" (2.590 m)	0.53	0.67	0.47
9'0" (2.740 m)	0.46	0.57	0.40
Note: Moment factors may be interpolated for other tie lengths.			

Material Properties of Concrete and Prestressing

The material properties of concrete and steel undoubtedly play an important role in the design of a prestressed concrete tie (Lutch, 2009). For prestressed concrete (PSC) tie design, AREMA in section 4.4.2 references the American Concrete Institute (ACI) Building Code Requirements for Structural Concrete and Commentary ACI 318 to obtain specifications on both concrete and prestressing material properties as well as design procedures (AREMA, 2009 and Lutch, 2009).

Table 2.19: Values of moment factors for monoblock ties related to tie length [AREMA Sec. 4.4.1.2, 2009]

Type of stress	Maximum permissible stress (Psi)	
	Australia Standards *	AREMA (ACI 318-08)
Compression	$0.6\ f_c'$	$0.6\ f_c'$
Tension (flexure)	-	$0.25\ (f_c')^{0.5}$ - $0.5\ (f_c')^{0.5}$

Table 2.20: Maximum permissible bending stress in the concrete under the working conditions (after allowing for all losses of prestress) [AS1085.14, 2003 and ACI, 2008]

Type of stress	Maximum permissible stress (Psi)	
	Australia Standards	AREMA (ACI 318-08)
Compression	$0.45\ f_c'$	$0.45\ f_c'$
Tension (flexure)	$0.4\ (f_c')^{0.5}$	$0.62(f_c')^{0.5}$
Where, $f_c' =$ concrete strength Note: Other important properties of concrete and steel are tabulated in Appendix-C, Table C-4.		

Table 2.21: The maximum allowable permissible stress in the prestressing tendons [AS1085.14, 2003 and ACI, 2008]

Stress case description	Allowable Prestressing Stresses (ksi)		
	Australia Standards	AREMA (ACI 318-08)*	
Due to prestressing steel jacking	$0.8\ f_{pu}$	$0.8\ f_{pu}$	$0.94\ f_{py}$
Immediately after prestress transfer	$0.7\ f_{pu}$	$0.74\ f_{pu}$	$0.82\ f_{py}$
Where, f_{pu} = ultimate tensile strength of wire or strand tendons and f_{py} = yield tensile strength of wire or strand tendons Note: (*)- for low relaxation wire and strands, $f_{py} = 0.9\ f_{pu}$			

The tensile stresses in steel prestress shall not exceed the lesser $0.8\ f_{pu}$ or $0.94\ f_{py}$ for stresses due to prestressing steel jacking force and similar for after prestress transfer (Lutch, 2009).

Design of Sleepers

The current international design standards for PSC sleepers, according to Remennikov et al (2008), are based on the permissible or allowable stress of materials concept which limits the strengths of materials to relatively low values compared to their true capacity. In addition this approach does not consider the probabilities of actual loads, risks associated with failure, and other factors which could lead to overdesigning the PC sleepers. Instead various limiting values or reduction factors are applied to material strengths and load effects. According to Krishna et al (2006) the maximum bending moment resistance (cracking moment) depends on the pre-compression, modulus of rupture of concrete and the section modulus and can be calculated by the following equation:

$$M_R = Z(P + f_r) \qquad (2-51)$$

Where: M_R= Moment resistance; Z = Section modulus; P = Pre-compression due to transfer of prestress to concrete; and f_r = Modulus of rapture of concrete.

According to Lutch (2009) unlike typical prestressed concrete design, flexural failure in ties (i.e. the primary limit state) is not defined by steel rupture or concrete crushing. Instead failure as defined by AREMA (Section 4.9.1) is the propagation of cracks from the extreme tension fiber of the tie to the first layer of prestressing (AREMA, 2009). On the other hand according to Remennikov et al (2008) the collaborative research between the University of Wollongong (UoW) and Queensland University of Technology (QUT) has addressed the development of a new limit states design concept instead of the current permissible stress approach of code AS 1085.14, which the limit state is taking care of the realistic loading conditions and the true capacity of the sleepers for PSC sleepers design for Australia railway.

Limit states according to Remennikov et al (2008) can be either ultimate limit state, which correspond to the maximum load-carrying capacity or, in some cases, to the maximum applicable strain or deformation; or serviceability limit state, which concerns the normal use.

According to limit state design approach, stresses at transfer and stresses at service can be computed and compared to allowable concrete stresses (Lutch, 2009). At transfer the prestressing is released and the force transferred to the concrete whereas at the service load limit state the tie is subjected to the prestressing force minus all losses, both instantaneous and time-dependent which have occurred up to that time and the applied loads from the train and ballast.

Australia recommends a prestress loss of 25 percent for PSC sleeper; and for certain types of sleepers, a lower prestress loss has been proved with values of 22 percent being adopted in the final design (AS1085.14, 2003). Whereas ACI 318 (2008) allows a prestress loss to consider elastic shortening of concrete, creep of concrete, shrinkage of concrete, relaxation of prestressing steel stress, prestressing steel seating at transfer and friction loss.

2.4.4. 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 (Kaewunruen and Remennikov, 2008). In this case the rail is only connected to the immediate base plate.

The important functions of fastenings according to Selig and Waters (1994) are to retain the rail against the sleeper and 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

Elastic fasteners provide vertical and longitudinal restraint as well as lateral; and are connected to the top of tie and press down on the top of the rail base and against the web of the rail [Selig and Waters, 1994; Selig, 2004]. They have four primary components and are designed to perform a specific function within the fastening system [Gutierrez et al, 2007].

- 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.

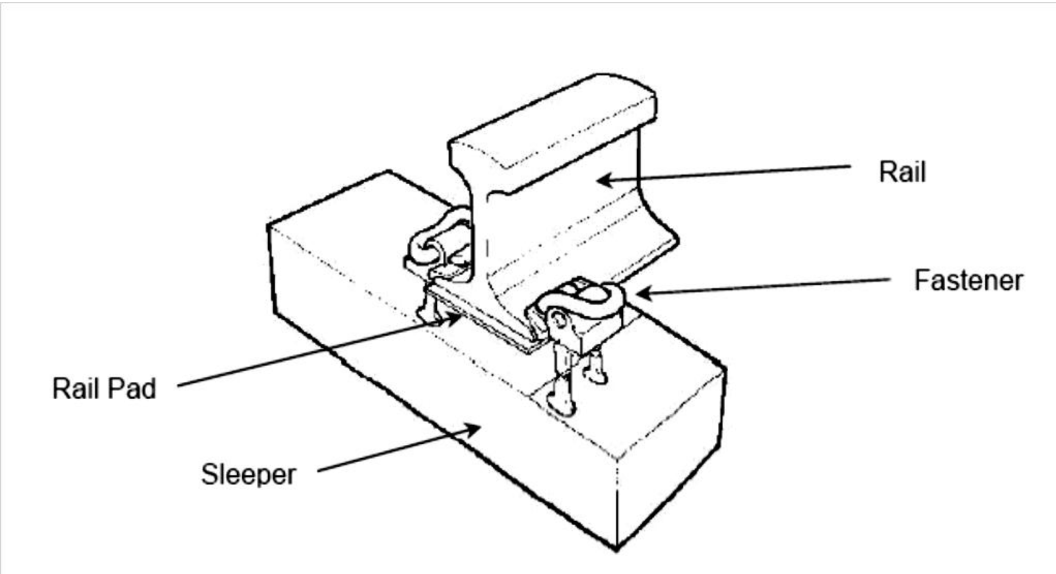


Figure 2.17: Typical fastening system for concrete sleepers [Kaewunruen and Remennikov, 2008]

Stiffness of Fastening Systems

The stiffness of a fastening system, expressed as the unit of applied force per unit of deflection (lb/in.), is one of the most important characteristics that directly impact the fastening system's long-term performance under repeated axle loading. Stiffness closely relates to the degree of wear fastening system components experience, and the resulting life of the system.

Selection of Fastener Types

Esveld (2001) states that the choice of fastening is greatly depend on the properties and structure of the sleeper. With the introduction of concrete sleepers, the spikes used by wood sleeper were replaced by double elastic fastening in which the rail is fastened by a spring using rubber pads (Miura et al, 1998). Spring clips are an integral part of the concrete tie system for ballasted track structures.

As Miura et al (1998) indicates leaf springs are used in Japan primarily because of cost and adjustability and in France because of fastening force and bearing ability; and wire springs (e.g. Vossloh) are preferred in Germany because of fastening force and adjustability. But according to Railway Group Standard (GC/RT5013, 1995), 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.


(a) (b)


(c) (d)

Figure 2.18: 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 [Miura et al, 1998; CED-AAU, 2010]

Rail pads are required between the rail seat and concrete sleeper surface to provide sufficient resiliency for the rail/sleeper system; provide damping of wheel induced vibrations; prevent or reduce rail/sleeper contact attrition; and provide electrical insulation for the track signal circuits [Selig and Waters, 1994]. In addition the pads and fasteners permit limited vertical deflection and rotational movements of the rail relative to the sleeper [Pen, 2008]. Resilient pads are not used with wood sleepers because the wood itself provides resiliency.

Performance Test on Fastening System

Forces acting on the fastening system are vertical, lateral, rotational (both planes), and longitudinal; and are the result of repeated loading cycles from passing axles, as well as longitudinal stresses in the rail. Fastening systems components are constructed from a variety of materials (with variable properties) to securely attach the rail to the tie and properly attenuate and/or transfer loads (Steidl, 2007).

A performance tests are conducted for fastening systems to resist the effect of such acting forces and environment (Steidl, 2007). The most commonly tests on fastening systems practiced in Europe and North America are repeated load test, longitudinal load restraint, determination of clamping force, stiffness of rail pad test, electrical resistance and torsional resistance (AREMA, 2009; EN 13146-4, 2002).

- Repeated load test is performed to determine ability of fastening system to withstand vertical and lateral loads.
- Longitudinal load restraint is performed to determine ability of fastening system to resist longitudinal rail movement.
- Clamping Force determination is performed to determine the force which is necessary to separate the rail from the rail pad.
- Stiffness of rail pad test is performed to determine load-deflection properties of the tie pad.
- The electrical resistance is determined between two short lengths of rail to determine ability of tie and fastener system to resist conducting electrical currents under wet conditions.
- Torsional resistance, performed in European standard (EN 13146-4, 2002), is important for resistance against track buckling.

To summarize, fastener design issues and design criteria are based on satisfying the above performance tests (Steidl, 2007).

2.4.5. Track Ballast

Railway ballast layer, the upper stratum of the substructure which supports the rails and sleeper, is one of the most important components in the railway track infrastructure (Aursudkij, 2007).

The most important functions of ballast according to Selig (2004) and Indraratna et al (2006) are to distribute the load from the sleepers, to damp dynamic loads, and to provide lateral resistance and rapid drainage.

Ballast is the preferable choice for substructure material over other alternatives such as concrete slabs or asphalt. This is because ballast provides less stiff support (which is an important factor in case of differential settlement or subgrade failure), is more economical, and produces less noise [Aursudkij, 2007; Ionescu, 2004].

2.4.5.1. Ballast track component

As a component of railway track substructure (Figure 2.11), the ballast can be classified in four zones according to Selig (2004):

a. crib, or the ballast in between the sleepers;
b. shoulder, the material beyond the sleeper ends down to the bottom of the ballast layer;
c. top ballast, the upper portion of load bearing ballast layer which is disturbed by tamping; and
d. 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.

2.4.5.2. Ballast Material Specification

The chosen type of ballast material usually depends on the local availability. The most important ones are to resist vertical, lateral, and longitudinal forces applied to the sleepers and to provide resiliency and energy absorption for the track (Aursudkij, 2007; Ionescu, 2004). 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 (Selig and Cantrell, 2001).

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 [Bonnett, 2005; Esveld, 2001; Pen, 2008; Indraratna et al, 2006].

- 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.

Ionescu (2004) stated that freshly placed ballast is a narrow-graded material (i.e. contains a limited range of particle sizes typically between 13 mm and 63 mm), consisting of a large amount of open pore space and a permeable structure.

If ballast particles are larger than the maximum size stated there may only be two or three stones between the underside of the sleeper and above the sub-grade which will be insufficient to properly distribute the load. Too many small stones below 28mm will however clog the ballast and reduce, in the longer term, its drainage properties (Bonnett, 2005). The permissible percentage of fines, generally grain sizes with a diameter below 22.4 mm, in relation to the total weight of the sample for new ballast is generally 3 to 5 % (Wenty, 2007). In addition, grain composition determines the ballast properties and an excessive proportion of fines could impair, impede or even totally rule out certain desirable ballast properties.

To ensure that ballast is of good quality, ballast needs to be tested after the manufacturing process at the quarry. Railway engineers are mainly interested in mechanical and dimensional properties (Aursudkij, 2007). But at present according to Ionescu (2004), the railway authorities worldwide do not agree on which are the optimum ballast index characteristics (gradation, grains size, shape, hardness, abrasion resistance and mineral composition) that will provide the best track performance. In addition, 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 (Lim, 2004). Large voids are required in the ballast for storage of fouling materials and drainage of water falling onto the track (Lim, 2004).

2.4.5.3. Ballast specification practices

The gradation of ballast plays a significant role in the strength, deformation, degradation, stability, safety and drainage of tracks. A specified ballast gradation according to Indraratna and Salim (2005) must have the following two key objectives

- Ballast must have higher shear strength to provide increased stability and minimal track deformation. This can be achieved by specifying broadly-graded (well graded) ballast.
- Ballast must have high permeability to provide adequate drainage, hence readily dissipating excess pore water pressures and increasing the effective stress. This can be insured by specifying uniformly graded ballast.

These two objectives are contradictory in terms of required particle size distribution and the optimum ballast gradation needs a balance between the uniform and broad gradation. According to Unified Soil Classification System (USCS) cited by Selig and Waters (1994) broadly graded material is defined by the gradation curve shape factor such as coefficient of uniformity ($C_u = \frac{D_{60}}{D_{10}}$) greater than 4 and coefficient of curvature ($C_c = \frac{{D_{30}}^2}{D_{60} \times D_{10}}$) between 1 and 3. Based on the test finding of Indraratna et al (2001) cited by Indraratna and Salim (2005), they recommended a ballast gradation with a limited coefficient of uniformity, [$2.2 < C_u \leq 2.6$]. They (Indraratna and Salim, 2005) also added that a C_u value exceeding 2.2 decreases the extent of ballast material breakage; and from drainage point of view this gradation has sufficient permeability and is acceptable for track substructure as long as the ballast is free from fines (fouling).

(a) United Kingdom (UK)

In addition UK selects five ballast properties to define the specification for track ballast: ballast grading, Los Angeles Abrasion (LAA), micro-Deval attrition (MDA), flakiness index (FI), and particle length index (PLI) (BS EN 13450, 2002 cited by Lim, (2004)).

Table 2.22: UK grading specification for ballast particle size (BS EN 13450, 2002 cited by Lim, 2004)

Square Sieve Size (mm)	63	50	40	31.5	22.4	32-50
Percent passing BS sieve	100	70-100	30-65	0-25	0-3	50

Table 2.23: The recommended limit value of ballast specification properties of UK (BS EN 13450, 2002 cited by Lim, 2004)

Properties	LAA	MDA	FI	PLI
Limited value	≤ 20	≤ 7	≤ 35	≤ 4

(b) United States (US)

The grading of the processed ballast shall be determined with laboratory sieves having square openings conforming to ASTM specification E 11 [AREMA, 2009].

Table 2.24: Recommended ballast gradations of US [AREMA, 2009]

Size No.	Nominal size square opening	Percent Passing								
		3"	$2^1/_2$"	2"	$1\,^1/_2$"	1"	$^3/_4$"	$^1/_2$"	d"	No.4
24	$2^1/_2$"– $^3/_4$"	100	90-100	-	25-60	-	0-10	0-5	-	-
25	$2^1/_2$" – d"	100	80-100	60-85	50-70	25-50	-	5-20	0-10	0-3
3	2" –1"	-	100	95-100	35-70	0-15		0-5	-	-
4A	2" – $^3/_4$"	-	100	90-100	60-90	20-55	0-10	-	0-3	-
4	$1\,^1/_2$" – $^3/_4$"	-	-	100	90-100	20-55	0-15	-	0-5	-

The percent material passing on sieve No. 200 Sieve should be limited to 1% and a limit minimum bulk specific gravity of 2.6.

Table 2.25: The recommended limit value of ballast specification properties of US (AREMA Vol-1 Section 2.4, 2009)

Properties	Soundness	Flat and/or Elongated Particles	Clay Lumps & Friable Particles	Degradation	Absorption
Limited value (%)	≤ 5	≤ 5	≤ 0.5	≤ 35	≤ 2

(c) Australia

Ballast properties to define the specification for track ballast: ballast grading, Los Angeles Abrasion (LAA), particle shape, Durability (Aggregate Crushing Value – ACV and Wet Attrition Value - WAV), flakiness index (FI), and bulk and particle density.

Table 2.26: Railway Ballast Standard Gradation (AS1141 cited by ETA-04-01 Ver1.1, 2007)

Sieve Size (mm)	63	53	37.5	26.5	19	13.2	4.75	0.075
Percent passing	100	85-100	20-65	0-20	0-5	0-2	0-1	0-1

Table 2.27: The recommended limit value of ballast specification properties of Australia in percent (AS1141 cited by ETA-04-01 Ver1.1, 2007)

Properties	LAA	ACV	WAV	FI	Weak particle	Misshapen particle	Uncrushed aggregate
Limited value (%)	≤ 25	≤ 25	≤ 6	≤ 30	≤ 6	≤ 30	≤ 5

The compacted bulk density of ballast material shall not be less than 1200 kg/m3 and the particle density of ballast material shall not be less than 2500 kg/m3 (ETA-04-01 Version 1.1, 2007).

(d) Finland

The EN standard (EN 13450, 2002) assigns railway ballast to one of six (A-F) grain size distribution classes. According to Finnish Rail Administration's requirements (2005a), the ballast used on the main lines must belong to grain size distribution class F, meaning that at least 85 % of the grains falls between 31.5-63 mm (Nurmikolu, 2005).

Table 2.28: Finnish Rail Administration's mainline ballast requirements (Nurmikolu, 2005)

Sieve Size (mm)	80	63	50	40	31.5	25	12	1
Percent passing	100	94-100	45-70	15-40	0-7	-	-	0-3

The Finnish Rail Administration (2005a), cited by Nurmikolu (2005), limits the fines content (<0.063 mm) a ballast sample to 1.0 percent as defined by wet sieving and the amount of grain sizes greater than 100 mm should be less than 12%. In addition for the resistance of fragmentation, ballast can be divided into three classes on the basis of the Los Angeles coefficient ($LA_{RB}12$, LA_{RB} 16 and $LA_{RB}20$).

Table 2.29: The main requirement properties for railway ballast used in Finland in percent (Nurmikolu, 2005; Väisänen and Kaivola, 2002)

<table>
<tr><th>Class</th><th>Annual traffic</th><th>LAA</th><th>MDA</th><th>Nordic ball mill</th><th>Shape index</th></tr>
<tr><td>R1/R2</td><td>$\geq$ 9MGT</td><td>$\leq$ 12</td><td rowspan="3">$\leq$ 11</td><td rowspan="3">$\leq$ 14</td><td rowspan="3">$\leq$ 20</td></tr>
<tr><td>R3</td><td>$>$ 3MGT & $<$ 9MGT</td><td>$\leq$ 16</td></tr>
<tr><td>R4</td><td>$\leq$3MGT</td><td>$\leq$ 20</td></tr>
</table>

(e) Ethio-Djibouti Railway

The study on the rehabilitation of the old railway line of Ethio-Djibouti was done and contains the most relevant information on material investigation for ballast and sub-ballast (Consta JV, 2007). According to the study, it is indicated that ballast shall consist of crushed stone, and be free from clay, debris, and organic or other deleterious matter. In addition it shall be angular in shape with all dimensions approximately equal.

Table 2.30: Grading of crushed stone ballast particle (Consta JV, 2007)

Sieve (mm)	70	64	45	32	1
% passing	100	93-100	25-55	0-7	0-1

Table 2.31: Consta JV's suggestion of ballast requirements (Consta JV, 2007)

Parameter	Values
Density	≥1400 kg/m^3
LAA	< 22%
Crush resistance	120/ 140 N/mm^2
Aggregate Impact Value (AIV)	< 22%
Water Absorption Value	< 0.5%
Elongation Index (EI)	< 25%
Flakiness Index (FI)	< 25%
Contamination with fine material	< 1%

(f) Ethiopia Road Authority (ERA)

Even if the gradations of the granular pavement material (base course) are different from ballast gradation, the tests performed to characterize the soil have similarity since the base course uses crushed aggregate (ERA, 2002)

Table 2.32: Ethiopia road Authority of base requirements (ERA, 2002)

Properties	LAA (*)	ACV	AIV(**)	FI	Sodium Sulphate soundness	Water Absorption
Limited value	≤ 30	≤ 25	−	≤ 30	≤ 12	≤ 2

Note: (*) The limited value of LAA is according to different road projects of Ethiopia and
(**) The limited value of aggregate impact value is not given.

2.4.5.4. Ballast track problems

A number of serious track problems, including ballast fouling, ballast degradation, settlement, pumping, subgrade failure and excessive ballast breakdown, are related to the lack of adequate drainage of the ballast layer [Indraratna et al., 2006; Bonnett, 2005].

Ballast Degradation: Ballast particles can suffer degradation due to the action of traffic and maintenance operations in broadly two ways (Bonnett, 2005): Either edge can become rounded and lose their interlocking effect or particles can break or crush under repeated loading.

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 [Aursudkij, 2007].

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 [Ionescu, 2004].

Settlement: Settlement of ballast is influenced by large trainloads, the number of load cycles, and high speed of trains [Indraratna et al, 2006]. As identified by Selig and Waters (1994) settlement of ballast may not be a problem if it occurs uniformly along the length of the track.

2.4.5.5. Ballast Section Design

The track performance of railway mainly depends on the thickness of the ballast layer and track deformation and degradation characteristics of the corresponding layer [Ionescu, 2004]. Wenty (2007) also indicated that with the increase of axle loads, the quality of the ballast under the ties and the quality of the subgrade becomes extremely important. In addition according to Bonnett (2005), the required depth of good quality ballast beneath sleepers varies depending upon the maximum speed of trains, the maximum axle loads carried and the gross annual tonnage expected.

Table 2.33: Different countries standard ballast minimum depth limitation [AREMA, 2009; ETA-04-01 Ver1.1, 2007; GC/RT5014, 1995; and EN 13450, 2002 cited by Krishna (2006)]

Railway System	Ballast depth (mm)
Australia	200-300
UK	225-375
USA	300
Sweden	240
RGS	300

Economic Section of Ballast

The generally accepted railroad practice of United States limit subgrade pressure to 25 psi and the following equations have been developed to determine the depth of ballast (h) required from the desired pressure [Selig and Waters, 1994; AREMA, 2009].

a. Talbot Equation:

$$P_c = \frac{16.8 * P_m}{{h_b}^{1.25}} \qquad (2-52)$$

Where: P_c = Maximum intensity of pressure on subgrade (25 psi)

P_m = Intensity of pressure on ballast = (psi)

$P_m = 2 * q/A_b$, q is rail seat load and A_b is tie-ballast contact area

h_b = Depth of ballast below tie (inch)

b. Japanese National Railways Equation (for meter gauge):

$$P_c = \frac{50 * P_m}{10 + {h_b}^{1.25}} \quad (2.53)$$

Where h_b = Depth of ballast below tie (centimeters)

c. Boussinesq Equation:

$$P_c = \frac{6 * q}{2\pi * {h_b}^2} \quad (2.54)$$

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. A value for BSW of not less than 12 inches is recommended for standard gauge construction of continuous welded rail (CWR) in main track service (AREMA, 2009). The ballast shoulder widths at the ends of ties in each direction for tie spacing T_s-inch are as follows:

$$BSW = \frac{P * T_s}{2 * P_v} \quad (2.55)$$

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 is calculated by the following equation:

P = 0.441D ΔT, where: D = Degree of curve (°) and ΔT = Temperature change

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, 2009).

2.4.6. Track Subballast

The subballast is the second structural layer of granular material that has the main functions to distribute stress from ballast to subgrade. In addition 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 [Selig and Cantrell, 2001; Aursudkij, 2007; Selig, 2004]. In addition it keeps the subgrade from penetrating up into the ballast while wet and under pressure [Boras, 2004].

Filter fabric (geo-textile) has frequently been used as subballast to fulfill track performance, but investigations have shown that fabric is not generally desirable. Asphalt concrete is an alternative or supplement to sand/gravel subballast materials, but the economics will probably limit asphalt to special cases (Selig and Cantrell, 2001).

2.4.6.1. Sub-ballast Specification

The subballast should be small particles of a material (finer than ballast) that will not disintegrate. Stone or slag screenings, chat (residue after extracting ore from rock), and sand make acceptable subballast [Boras, 2004].

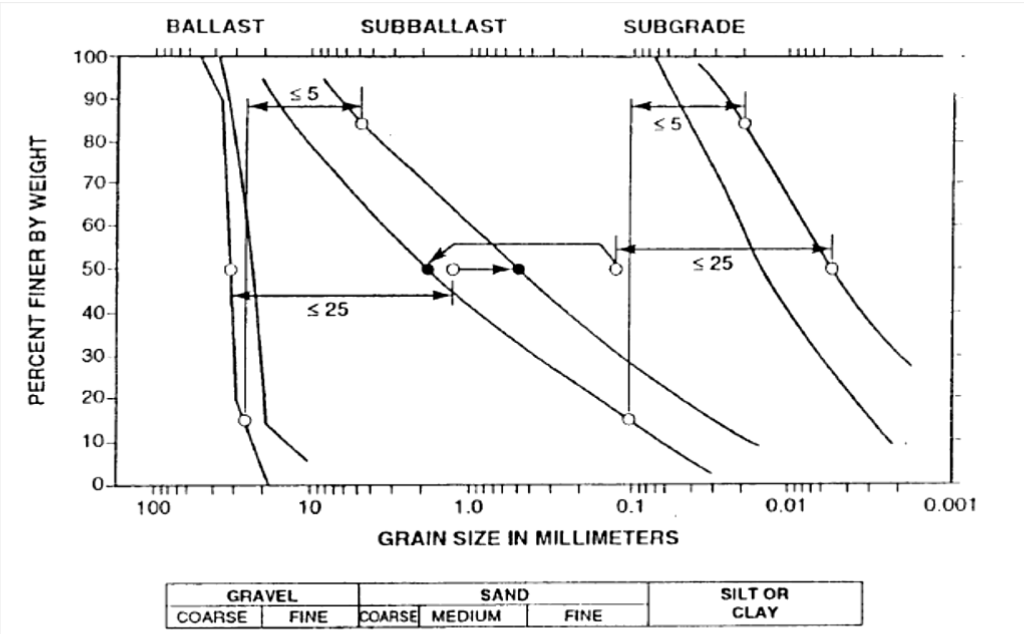


Figure 2.19: Subballast satisfying filter criteria (Selig and Waters, 1994)

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 [Lim, 2004].

The durability requirements of subballast are not as severe as for ballast because the subballast particles are smaller and the stresses are lower [Selig, 2004].

To provide separation between the ballast and subgrade particles, the subballast gradation must satisfy the requirement in Figure 2.18. This provides that the finest subballast particles are smaller than the largest subgrade particles, and correspondingly the largest subballast particles must be larger than smallest ballast particles. There must also not be gaps in the gradation of the subballast. Subballast satisfying these requirements will also be satisfactory for preventing attrition on the hard subgrade surface by the ballast [Selig, 2004].

US Practice

Material most commonly available for use as sub-ballast are those aggregates ordinarily specified and used in construction for highway bases and subbases (AREMA, 2009). These include crushed stone, natural or crushed gravels, natural or manufactured sands, crushed slag or a homogeneous mixture of these materials.

According to AREMA standard recommendation, the gradation of sub-ballast material can be determined by use of the grain size distribution of the sub-grade by applying filter principle.

Table 2.34: Requirements for filter material [AREMA, 2009]

Character of filter materials	Ratio R_{50}	Ratio R_{15}
Uniform grain-size distribution (Uniformity coefficient C_u=3 to 4)	5 to 10	-
Well graded to poorly graded (non-uniform); sub rounded grains	12 to 58	12 to 40
Well graded to poorly graded (non-uniform); angular particles	9 to 30	6 to 18

Where: $R_{50} = D_{50}$ of filter material/D_{50} of material to be protected
$R_{15} = D_{15}$ of filter material/D_{15} of material to be protected

Note: - Grain-size curves (semi-logarithmic plot) of sub-ballast and subgrade should be approximately parallel in the finer range of sizes.
- This table was prepared especially for earth dam design and since the use here is for a railway the values given may be slightly exceeded.

The grain-sizes at 15%, and 50% points (grain size corresponding to 15% and 50% percent passing) of the sub-grade material are first determined. These values with the relevant ratios, from Table 2.34, can be used to compute the limiting grain sizes at the 15% and 50% passing lines of the sub-ballast material. No more than 5% of the sub-ballast should pass the No. 200 sieve [AREMA, 2009].

The most frequently used tests for sub-ballast material are: particle size analysis; moisture density relation (MDD and OMC); Atterberg limits; degradation–LAA; Sodium Sulphate Soundness; permeability; percent material passing no. 200 sieve; and specific gravity. The test methods are according to ASTM with respective procedure. The limiting values are determined by characteristics of the ballast and subgrade as well as the material used for sub-ballast.

Finland

Finnish Rail Administration (FRA) requires a uniformity coefficient $C_u \geq 6$ for crushed rock aggregate and the fines content (< 0.063 mm) of the subballast crushed rock aggregate is limited to maximum of 3%. The FRA sets requirements for the modified Swedish impact value (<22) and Nordic ball mill value (<16) of crushed rock aggregate used in track substructure (FRA, 1999).

Table 2.35: Requirements of FRA for crushed rock aggregate of intermediate and frost protection layer (Nurmikolu, 2005)

Sieve Size (mm)	63	50	40	31.5	16	8	2	0.1
Percent Passing	100	83-100	66-100	52-100	23-89	0-55	0-6	0-4

Ethio-Djibouti Railway

The sub-ballast must consist of well graded sandy gravel, with a small percentage of fine elements. Grading and property requirements of crushed aggregate for sub-ballast shall be in accordance with the following tables according to Consta JV's suggestions.

Table 2.36: Consta JV's suggestion of sub-ballast requirements (Consta JV, 2007)

Parameter	Recommend values
Organic content	< 0.2% of the weight of the material fraction passing sieve size 2 mm (UNE 103 – 204: 1993)
Sulphate content	< 0.2% of the weight of the material fraction passing sieve size 2 mm (UNE 103 – 201: 1993)
LAA	< 28%
Water Absorption	< 22%
CBR	> 20 at 95% of modified AASHTO density
Permeability	$\leq 10^{-6}$ m/s

Table 2.37: Consta JV's suggestion of gradation for sub-ballast requirements

Sieve size (mm)	40	31.5	16	8	4	2	0.5	0.2	0.063
% passing	100	90-100	85-95	65-80	45-65	30-50	10-40	5-25	3-9

Note: the parameters to be satisfied along the gradations are; Coefficient of Uniformity, $C_u \geq 14$ and Coefficient of Curvature, $1 \leq C_c \leq 3$.

Ethiopia Road Authority (ERA)

The recommended particle size distributions for suitable materials (GB1, GB2 and GB3) in Ethiopia road construction for nominal size of 37.5 mm are tabulated below.

Table 2.38: Gradation requirement for base course material (ERA, 2002)

Sieve size (mm)		50	37.5	20	10	5	2.36	0.425	0.075
Percent passing	GB1	100	95-100	60-80	40-60	25-40	15-30	7-19	5-12
	GB2 & GB3	100	80-100	60-80	45-65	30-50	10-40	10-25	5-15

Note: The granular base course (GB), GB1- is fresh and crushed rock; and GB2 & GB3- are naturally occurring granular materials, boulders, and crushed weathered rocks.

2.4.6.2. Sub-ballast Depth (h_{sb}) Design

Almost all leading world railways provide a layer of sub-ballast along with ballast. However, there is wide variation in the practices followed in different countries of the world (Krishna, 2006). According to Selig (2004) and Krishna (2006), the minimum subballast layer thickness (h_{sb}) beneath the ballast, according to AREMA (2009), should be 6 inches (150 mm) even though it is not required from sub-grade stress point of view. In addition the Australia railway provides a minimum subballast depth of 150 mm and UK provision is variable (Krishna, 2006).

The distribution of loads to depth is approximately the same regardless of the granular material. Therefore the combined depth of sub-ballast and ballast is calculated as a single unit to develop the pressure on the subgrade. The minimum depth of ballast (ballast + subballast) in inches required to produce a stable structure is therefore from equation (2.52) according to Talbot empirical formula for vertical pressure exerted by the ballast under the tie:

$$h_b = \left[\frac{16.8 * P_a}{P_c}\right]^{4/5} \tag{2.56}$$

Where: P_c = bearing pressure on subgrade including safety factor (psi)

P_a = uniformly distributed pressure over tie face (psi)

h_b = Depth below face (inch)

The side slope run component of the Sub-ballast Section is proportioned to provide drainage from the top of the roadbed construction. A value for side slope shall not be less than 24 or more than 40 is recommended. Sub-ballast materials having relatively lower permeability rates may use relatively higher side slope values (AREMA, 2009).

2.4.7. Subgrade Soil

The railway subgrade is the load-bearing layer of a track structure, either compacted natural ground or an imported fill embankment, which provides a permanent way to support the track bed (ballast and sub-ballast layers) [Radampola, 2006; Selig, 2004].

The influence of the traffic-induced stresses extends downward as much as 5 meters below the bottom of the ties (Selig, 2004). This is considerably beyond the depth of the ballast and subballast. This deep layer must have sufficient bearing capacity, provide good drainage and yield a tolerably smooth settlement in order to prolong track serviceability under specific operating and climatic conditions [Kaewunruen and Remennikov, 2008; Piotrowski, 2007].

The main functions of subgrade according to Selig (2004) are to: provide a stable platform to construct the track; limit 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 is placed between the natural ground or the embankment fill material and the ballast to improve the capacity of the track structure and to minimise its ongoing costs of maintenance, especially issues related to track geometry (Radampola, 2006).

2.4.7.1 Subgrade Failures

Subgrade failure usually occurs due to over stressed conditions, poor construction or maintenance practices such as inadequate foundation preparation or inadequate compaction or excessive moisture content of the filling material. Similarly, natural conditions such as weak subgrade soil (silt and clay), high ground water tables and erosion or sliding of embankment also affect the performance of the subgrade (Radampola, 2006).

Figure 2.20: Bad track quality due to soft subgrade [Wenty, 2007]

The subgrade to serve as a stable platform, the failure types must be avoided. The most commonly used remedial measure, which is considered the most economical is to increase the ballast thickness (ballast + subballast) to reduce subgrade stresses. The other possible remedial measures are constructing new track, increased rail weight, using a HMA layer and modifying subgrade to permit higher stresses (Radampola, 2006).

2.4.7.2. Problematic Soils in Ethiopia

According to Low volume roads (LVR) manual of ERA (2011) a number of subgrade materials fall into the category of "Problem Soils" and, when encountered, would normally require special treatment before acceptance in the pavement foundation. This category of soils includes:

a) Expansive clays- which exhibit particularly large seasonal volumetric changes (swell and shrinkage) and may be dark or light grey clay.
b) Collapsible sands- occur mostly in the arid and semi-arid regions of eastern and south eastern Ethiopia and have a low dry density and low clay content. They exhibit a weakly cemented soil fabric which may be induced to rapid settlement.
c) Dispersive (erodible) soils- are prevalent over many areas of Ethiopia
d) Saline soils- occur mostly in the arid or semi-arid regions of Ethiopia and cause salt damage.
e) Micaceous soils- are those soils which contain large quantities of mica (muscovite) and occur in such materials as weathered granite, gneiss, mica schist and phylite materials that occur in various areas of Ethiopia.
f) Low-strength soils- are soils with a soaked CBR of less than 3 per cent (< 2 per cent in dry climates). They occur particularly in the low-lying, swampy areas of Ethiopia.

2.4.7.3. 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 (SATTC, 1988 cited by Nadew Abdisa (2010)).

A properly selected material grain size distribution according to Piotrowski (2007) is assumed to meet the coefficient of uniformity greater than 5, i.e. $C_u = \frac{d_{60}}{d_{10}} \geq 5$. Where: C_u is the coefficient of uniformity, d_{60} – diameter of particles which with all the less ones represent

60% of soil mass, and d_{10} - diameter of particles which with all the less ones represent 10% of soil mass.

II. Atterberg limits

Atterberg limit 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. The quantitative figure of liquid limit and plasticity index are used as an indicator whether the soil is expansive or not. Expansive clays exhibit higher liquid limit and plasticity index upon change in moisture content (REFD, 1993).

2.4.7.4. Engineering Properties of Subgrade Soils

The suitability of soil for a particular use depends on one or more engineering properties of a soil response to that use [Piotrowski, 2007]. Two engineering properties are important to many types of engineering works and situations involving soils; volume change under an applied load-- compressibility and shear strength-- the resistance of soil to sliding of one mass against another.

A. 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 (ASTM D 1557; Zelalem Worku, 2010).

Compaction is the process of increasing soil density or unit weight to improve the engineering properties either of an existing soil or during the process of placing a fill, accompanied by a decrease in air volumes but usually no change in water content. The main outcomes being sought are to:

- 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.

The geotechnical properties of soil (such as swell potential, compressive strength, CBR, permeability, and compressibility etc) are dependent on the moisture and density at which the soil is compacted (type of soil and degree of compaction).

According to the ERA (2002) Pavement Design Manual, it is recommended that the top 25cm of all subgrades should be compacted to a relative density of at least 100% of the maximum dry density achieved by ASTM Test Method D 698 (light or standard compaction). Alternatively, at least 93% of the maximum dry density achieved by ASTM Test Method D 1557 may be specified.

B. Strength and stiffness properties

The basic subgrade stiffness/strength characterizations commonly used are California Bearing Ratio (CBR) and elastic (resilient) modulus. The strength and stiffness of materials is largely related to their compaction density, which depends significantly on the particle size distribution (Veisi et al., 2010).

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; typical ranges of CBR for different soil groups (according to USCS) are tabulated in Appendix C Table C-3 (Radampola, 2006).

Resilient Modulus (M_R) - 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.

2.4.7.5. Subgrade soil stabilization

Stabilization of subgrade is done for some soils when compaction will not produce the desired strength needed to support structures [Vises et al., 2010]. Benedetto (2010) stated that the technique of soil stabilization is usually adopted with the purpose of rendering plastic soils coherent to the standards and requirements of engineering projects. The effect of stabilization is quantified by a significant increase in the Atterberg limit and so by a relevant reduction of plasticity (Plastic Index).

Soil Stabilization Methods

The two frequently used methods of stabilizing soils are stabilization by mechanically or stabilization by chemical additives [Vises et al., 2010; Benedetto, 2010].

The use of lime, cement and bitumen along with mechanical stabilization and blending are traditional, accepted and proven approaches in the road engineering in Ethiopia [Alemayehu Ayele, 2010].

In the case of mechanical stabilization, rolling is the simplest and most commonly used method and is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification [Veisi et al., 2010; SUDAS, 2010]. In addition geosynthetics (a class of geomaterials) is a method of mechanical stabilization that is gaining popularity throughout the commonwealth and the types of geosynthetics that are most likely used for stabilization are geotextiles, geogrids, geocomposites, and geomembranes [Ashmawy and Bourdeau, 1995; SUDAS, 2010].

Mixing soils with stabilizing agents (additives or chemicals) like lime and cement, usually in low amounts, changes both the physical (OMC and the MDD) and the chemical properties of the stabilized soil [Piotrowski, 2007; Veisi et al., 2010].

Different chemical additives, practiced in Ethiopia, are marketed internationally as soil compaction aids and stabilizers; such as CON-AID Chemical, Dallas Roadways Product (DRP), PURE CRETE and TerraZyme [Alemayehu Ayele, 2010]. In addition Any-Way's Natural Soil Stabilization is implemented in more of AACRA's road projects, bringing an economic savings and environmental advantages to these projects (Zevid, 2009).

Guidelines for Stabilizer Selection

Soil characteristics including mineralogy, gradation and physio-chemical properties of fine grained soils influence the soil-additive interaction (Little and Nair, 2009). Hence stabilizer selection should be based on the effectiveness of a given stabilizer to improve the physio-chemical properties of the selected soil.

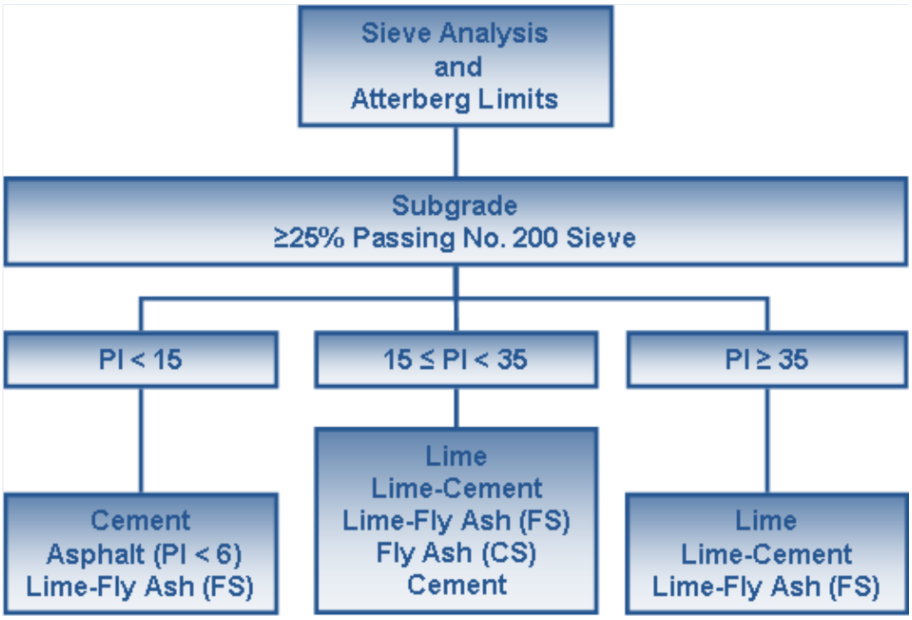


Figure 2.21: Additive selection criteria for subgrade using soil classification (Veisi et al., 2010)

The preliminary selection of the appropriate additive(s) for soil stabilization according to Little and Nair (2009) and Veisi et al. (2010) should consider:

- Plasticity Index (PI) and grain size distribution (gradation)
- Soil mineralogy and composition
- Purpose of treatment and desired engineering properties
- Mechanisms of stabilization
- Environmental conditions and engineering economics

2.4.7.6. Subgrade Design

To calculate the vertical pressure on the subgrade layer resulting from sleeper loading, the load transfer through the ballast layer should be investigated [Radampola, 2006].

Talbot Equation

The most commonly used empirical relationship to limit the subgrade stresses is the AREMA recommendation of Talbot Equation, developed in 1991.

$$P_c = \frac{16.8 * P_m}{h^{1.25}} \tag{2.52}$$

Where: h = granular layer thickness or (ballast + subballast) (in), P_c = allowable subgrade pressure (psi), and P_m = vertical stress applied on the ballast surface (psi).

Since the subgrade is often the weakest substructure layer, a combined of ballast and subballast thickness (track support layer) is required that will reduce the pressure on the subgrade to a level that produces an acceptably small deformation from the repeated train loading for the desired design life.

The important dimensions to be considered in roadbed design are the top width of the roadbed, the height of fill (or depth of cut) and the side slopes of the cut or fill section. In addition appropriate subsurface drainage system should be installed to lower the ground water level (Boras, 2004; SUDAS, 2010).

The top width of the subgrade must accommodate the track and ballast and may need to provide a walkway area outside of the ballast section. The top of the roadbed may be sloped downward away from the centerline to facilitate drainage from the ballast. The base of the roadbed must be wide enough to transmit the track and train loading within the allowable pressures of the natural ground material (Boras, 2004). According to Selig and Waters (1994), a more logical approach to designate allowable subgrade bearing pressure is based on the soil condition.

Capping layer (or an improvement layer) is a higher strength and higher stiffness layer introduced to protect weak natural ground or embankment fill by using a granular material (Radampola, 2006).

2.5. Railway Safety System

The railroad is still undeniably one of the safest modes of transport in levels of safety compared with the other mode of transport. However, as complex systems, despite the high technical and organizational effort to prevent accidents, sometimes such accidents happen with catastrophic consequences. It is recognized that such accidents rarely have simple causes and behind the obvious or direct causes, such as human errors or indirect concealed causes [Braband, 2007]. The main objective of the safety work is therefore to avoid these causes, if possible, or to control them. Improving track and equipment are the main technological improvements to railroad safety [Sussman and Raslear, 2007].

2.5.1. Railway Accident

Accident is an occurrence in the course of working of railway which does or may affect the safety of the railway, its engine, rolling stock, permanent way and works, fixed installations, passengers or servant or which affect the safety of others or which does or may cause delay to train or loss to the railway.

Types of railway accident are

- Derailment- may be at plain track, curves or junctions.
- Collision- may be head-on, rear-end, and grade (level) crossing
- Fire and violent eruption (including sabotage or terrorism) and
- Others (falls from train, collision with people on tracks)

The most causes can be divided into three groups

a. Inadequate technical activities
b. Ineffective communication and organization (management),
c. Weaknesses in safety culture.

Example of such causes include

- Mechanical problems (signal failures, equipment failures and train control)
- Human factors (train operation)
- Infrastructure defect (track, roadbed and structure)
- Environment or external factors (weather-related, obstruction)

A. Safety issues related to basic railroad infrastructure

Derailment is one of the most catastrophic accidents in railway operation. It could cause causalities and fatalities, and result in serious damage to track and train [Zhao et al, 2006]. Among all track components, rail breaks represent a major cause of railway derailment [Zhao et al, 2006]. Among the most common causes of derailments, bearing-failure is the less severe. Misalignment of railway track is another cause that leads to derailment and track is misaligned due to a combination of several factors. These factors can be summarised as: track disturbance by resurfacing, ballasting, cleaning, etc; track incorrectly adjusted; insufficient ballast depth and fouling; defect of sleepers and fastenings; insufficient superelevation or sharp "kink" in curves; trains exceeding speed boards; and lack of track maintenance.

Figure 2.22: The fractured rail in-situ [Zhao et al, 2006]

B. Safety issues related to human factors

Human error plays a part in most accidents, if not all. In critical systems like transport systems, safety measures against human errors play a substantial role. In railway operation, several safety-critical tasks are assigned to the operators and are not controlled by signaling and interlocking systems [Kumar and Sinha, 2008]. Human factors such as trespassing, failure to abide by signal warnings, driver and train operator negligence are common for train accident. Sources indicated that [European Railway Agency, 2010] most of the passenger fatalities occur when passengers try to embark or disembark trains that are moving as well as the major part of the number of fatalities occur due to unauthorized persons being hit by rolling stock in motion or level crossing accidents.

Fatigue and illness, a cause of serious accident, are issues that are not easily controlled by the wider transport sector and are more dependent on individual drivers and their employers [Peterman, 2007].

C. Safety issues related to environment or external factors

Obstruction and weather condition are external factors that affect railway safety system. Obstruction- when rail track are being blocked by debris from rock slides, mudslides, avalanches, fallen trees and similar objects. Weather- over half of all derailments were most closely associated with extreme heat, antecedent rainfall, and antecedent snow and ice conditions.

2.5.2. Highway-Rail Level Crossing Safety

Level crossings are well known components of railway networks with the greatest risk of collision and possibly derailment. Collisions between trains and highway vehicles are the second-leading cause of rail-related fatalities, after trespassing. These collisions occur primarily at places where roads cross railroad tracks at the same level or “at grade” [Peterman, 2007].

Figure 2.23: highway-railway level crossing safety system [Peterman, 2007]

In order to improve the safety of railway level crossings, it is first important to identify the causes of collisions. At the site at which two modes of transport meet, there are, of course, significant inherent dangers. While there are many underlying factors which have led to recent collisions at level crossings, almost every time the primary factor in the accident was the failure of the motorist to abide by the traffic control measures at the crossing [King et al, 2009].

In addition there are a number of other factors which are largely based on the failure of a motorist to detect an approaching train. Such that unclear line-of-sight for the road-user along the rail line; motorists’ awareness of trains is also impacted on by the design or engineering of certain level crossings; lack of visual properties on the train other than its standard headlight [King et al, 2009].

2.5.3. Railway safety management system

Complex chain of multiple events and deficiencies lead to accidents and catastrophes in railway transport system. The Railway Safety measure is therefore necessary to alleviate the problem, reduce fatal accidents and the socio-economical impact [European Railway Agency, 2010].

Rail transport is characterized by two ruling system properties: long stopping distances due to low friction as well as track guidance. Both properties have certain responsibilities in securing the rail transport from rail cars to follow concrete protection functions [Maschek, 2007]. Therefore special precautions are necessary against collisions with different types of objects. Major safety issues can be seen into two broad categories [Maschek, 2007, Braband 2007], i.e., railway system and functional safeties.

Risk and accident assessment

Some risks can be foreseen and readily addressed, whilst others are either so unexpected as to be unforeseeable or have only a very remote chance of occurrence. Risk can be reduced through investment in infrastructure, increased staffing, and better organization, operating practices and training. Some of these can be undertaken at low cost, and will readily result in the community being better off in net terms as a result [Bray, 2004].

Railway safety program

The development of a railway safety assessment program is reliable technique to audit safety documents automatically and to assist railway safety assessors with checklists and guidelines [Noh et al., 2010]. According to Noh et al., railway safety assessment may be classified as safety management system (SMS) and engineering safety management (ESM).

A safety management system (SMS) is a formal framework for integrating safety into day-to-day railway operations and includes safety goals and performance targets, risk assessments, responsibilities and authorities, rules and procedures, monitoring and evaluation processes (Bourdon, 2005). It is an important way of demonstrating that railway undertakings and infrastructure managers are operating and maintaining their part of the railway systems [European Railway Agency, 2010].

The engineering safety management (ESM) demonstrates compliance with the rail industry safety standards and provides stakeholders that the system designed and the installed will meet functional and safety requirements. Railway operation companies manage engineering railway safety activities to achieve their safety goals [European Railway Agency, 2010].

2.6. Station platform

Train stations comprise of the building(s), site access, parking, tracks, platforms, and all appurtenances necessary to conduct transportation [AREMA, 2009]. Station platform is a raised structure or area within a station providing access to or from a train (boarding or alighting). Station platforms shall be located on straight track unless the particular geographical characteristics of the site and the characteristics of the railway at the proposed location of the platform do not provide a reasonable opportunity for achieving this.

Station platforms shall not be located on horizontal curves with radii less than 1000m. It shall be located on track gradients not steeper than 1 in 500 wherever reasonably practicable.

From the experience of railway group Standards [GI/RT 7016, 2010], for new platforms, the height at the edge of the platform shall be 915 mm (with a tolerance of +0, -25mm) measured at right angles to the plane of the rails of the track adjacent to the platform. Whereas according to China National Standard of Code for Design of Railway Station and Terminal [GB 50091-2006, 2006], the recommended height at the edge of the platform for passenger train are 300 mm for low platform, 500 mm for ordinary platform and 1250 for high platform; and for freight platform are 1100 mm for ordinary platform and not more than 4800 mm for high platform. AREMA also recommends a minimum height at the edge of the platform for passenger boarding as 210 mm and 1310 mm for low and high level platforms respectively.

In addition, according to railway group Standards [GI/RT 7016, 2010], for most rolling stock, the minimum offset distance of the platform edge from the adjacent track shall be 730 mm (within a tolerance of +15, -0 mm). Whereas according to China design standards a minimum recommended offset distance of the platform edge from the adjacent track are 1750 mm from center line of the track except 1850 mm for freight high platform. AREMA also recommends a minimum value for passenger boarding as 1680 mm and 1700 mm for low and high level platform respectively from centerline of adjacent track to edge of platform.

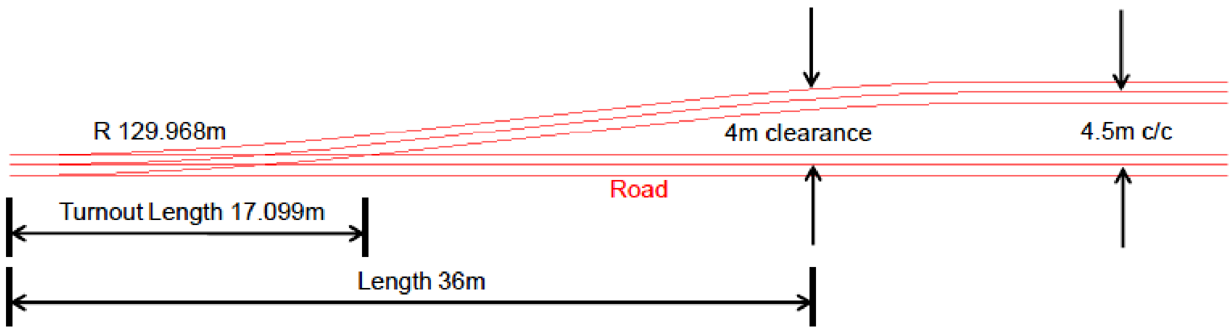
2.7. Track Switch, Turnout and Crossing

Switch is a pair of moveable rails with their fastenings and operating rods, providing a connection over which to divert the movement of rolling stock and other on-track equipment. Switches may be operated by power operated switch and lock movements; electrically locked hand-operated machines; or hand-operated trail able switch stands, depending on the location and purpose of the switch.

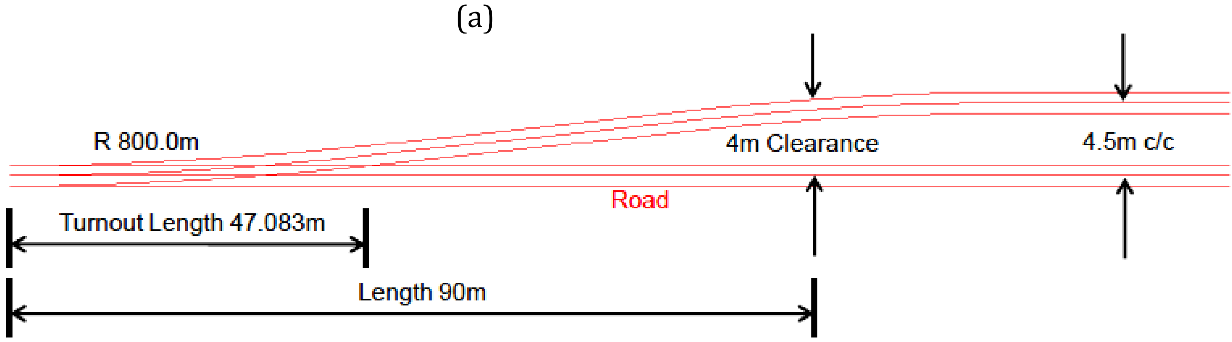
Turnout is a particular grouping of two tracks joined together with a frog and switch so arranged to allow for the transfer of rolling stock and on-track equipment to cross from one track to another.

For a turnout on main track, its rail type shall be identical with main track. The rail type of turnout on station track shall not be lower than that for the said track, if it is higher than the rail type for this track, it shall lay steel rail of the same type as the turnout or compromise rail for at least 6.25 m respectively in front and back of the turnout, this shall be no less than 4.5 m under difficult conditions, but not in a continuous manner [GB 50091-2006, 2006].

On the tracks where the train passing speed in straight direction is 160 km/h and above, it shall adopt simple turnout with movable nose frog. The turnout where train passing speed in straight direction is more than 120 km/h shall adopt separately movable external locking device. The turnout number also depends on the train speed and condition of the track [GB 50091-2006, 2006].



(a)



(b)

Figure 2. 24: Different turnout numbers (a) 1:7.52 turnout (b) 1: 18.5 turnout [Imrie P., 2009]

3. General Description of Ethiopia

Ethiopia is located between approximately 3^0-15^0 N latitude and 33^0-48^0E longitude. The country covers a land area of about 1.13 million km^2, occupying a significant portion of the Horn of Africa. The factors that shall be included for standardization of guidelines for railway infrastructure system of railway system of Ethiopia are discussed below.

3.1. Topography

Ethiopia is a country of great topographical diversity with altitudes ranging from 110 meters below sea level in the Dallol Depression (Kobar sink) to mountain peaks 4620 meters above sea level. It is made up of high and rugged mountains, flat topped plateau, and separated by the deep and steep sided gorges with rivers and rolling plains.

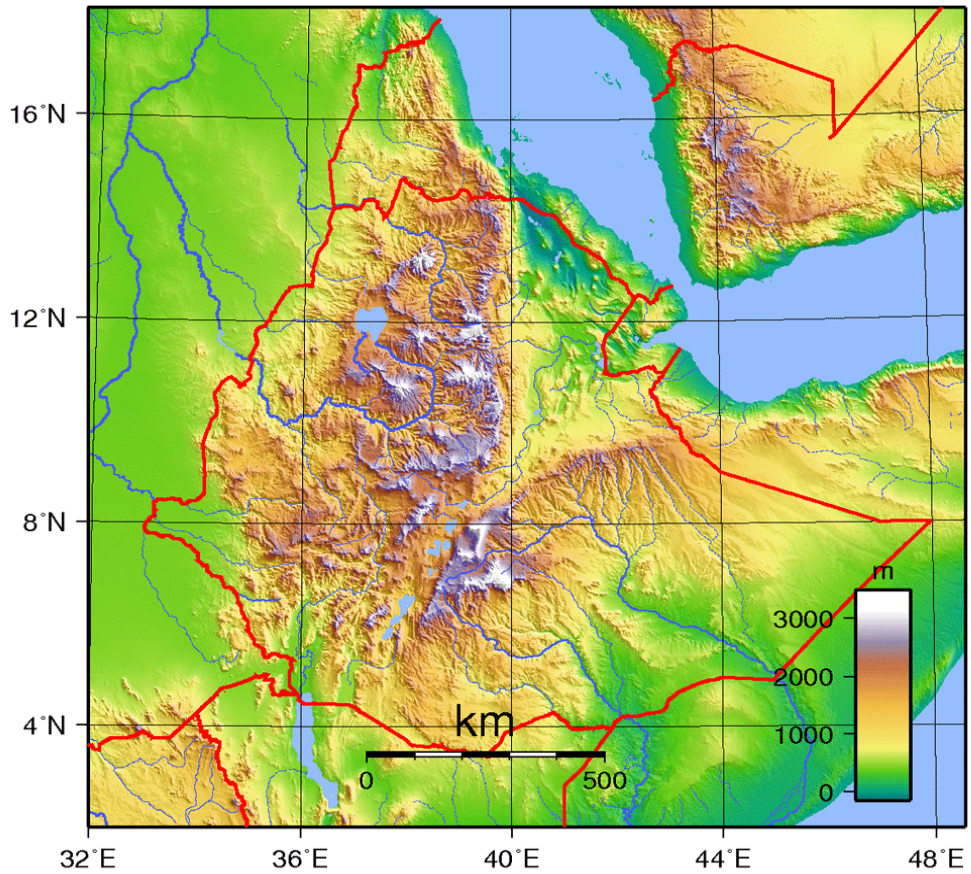


Figure 3. 1: Topographic map of Ethiopia [Sadalmelik, 2007]

The eastern margin of the plateau is elevated to between 3000m and 4000m, towards the West, while the plateau surface descends to between 1200m and 1000m. The Western highlands are massive with an average height of 2000-2500m. The western and the eastern highlands are divided by Rift Valley.

ERA Geometric Alignment Data

The topographic condition of Ethiopia in connecting different urban centers with the railway system can be easily understood using the corresponding data of road geometric alignment.

In 1996, the Ethiopian Roads Authority, ERA, invited TecnEcon Ltd to undertake a feasibility study of five roads that were being prepared for upgrading and/or rehabilitation as per Road Sector Development Program (RSDP). The five roads are:

- Modjo - Awash - Mille;
- Alemgena - Hossaina - Sodo;
- Woldiya - Adigrat - Zalambessa;
- Debre Markos - Gondar: and
- Awash - Kulubi - Dire Dawa - Harar.

The geometric alignment data of these projects such as rise and fall; and curvature are summarized in the Table 3.1.

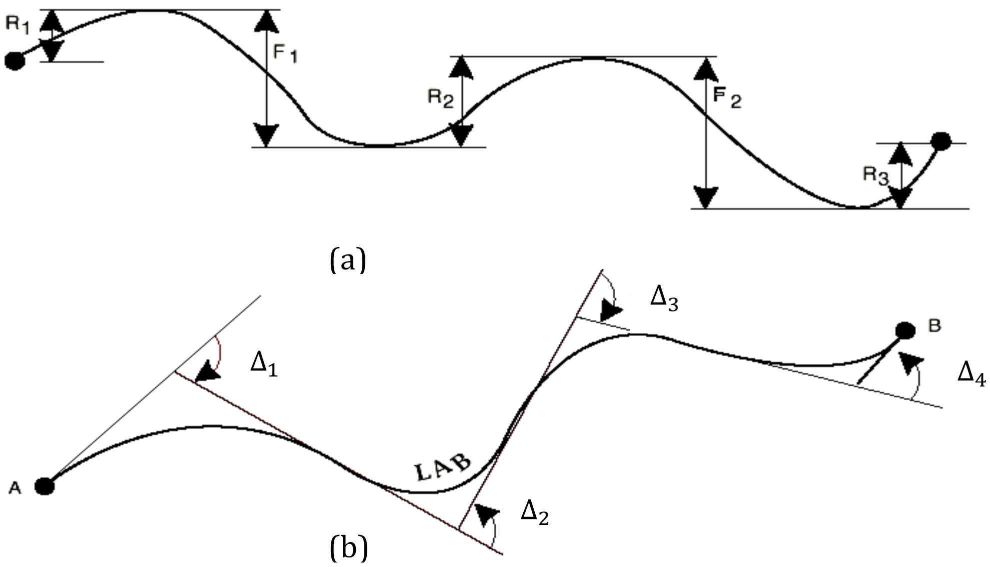


Figure 3. 2: (a) Vertical profile (rise and fall) and (b) Horizontal Curvature [Callao, 2008]

The rise and fall (m/km) and the average horizontal curvature (degrees/km) can be computed by the following two equations respectively.

$$\text{Rise plus Fall (m/km)} = (R_1 + F_1 + R_2 + F_2 + R_3)/\text{Length} \qquad (3-1)$$

$$\text{Horizontal Curvature (degrees/km)} = (\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4)/\text{Length} \qquad (3-2)$$

According to Table 3.1, there is significant section of roads with a rise and fall that leading a high gradient greater than 5% (50m/1km rise and fall) especially in the Northern part of Ethiopia. In addition using average horizontal curvature (degree/km), the radius or length of circular curve can be determined with the formula:

$$R = \frac{180 \times L}{\Delta \times \pi} \qquad (3-3)$$

Where: R- radius of the curve, L-length of circular arc between point of tangency and point of curvature, Δ-degree of curvature

Table 3.1: Geometric alignment data of different road projects [TecnEcon Ltd, 1996]

Road	Section	Surveyed Length (km)	Rise & Fall (m/km)	Avg. Curvature (deg/km)	Remark (Terrain)
Modjo-Nazret-Mille	Modjo-Nazret	18.6	17	117	Flat/Rolling*
	Nazret-Awash	129.1	13	80	Flat/Rolling*
	Awash-Mille	294.0	13	76	Flat/Rolling*
Alemgena - Hossaina - Sodo	Alemgena-Butajira	116.5	12	73	Flat/Rolling
		3.6	50	300	Hilly/Mountain
	Butajira-Hossaina	97.9	13	78	Flat/Rolling
		2.0	50	300	Hilly/Mountain
	Hossaina-Sodo	74.0	12	70	Flat/Rolling
		26.0	59	362	Hilly/Mountain
Woldiya - Adigrat - Zalambessa	Woldiya-Maychew	69.4	16	105	Flat/Rolling
		72.1	68	422	Hilly/Mountain
	Maychew-Mekele	24.8	20	150	Flat/Rolling
		87.7	69	426	Hilly/Mountain
	Mekele-Wikro	31.4	18	125	Flat/Rolling
		11.7	64	396	Hilly/Mountain
	Wikro-Adigrat	52.2	18	125	Flat/Rolling
		19.3	64	396	Hilly/Mountain
	Adigrat-Zalambessa	27.6	15	104	Flat/Rolling
		10.7	68	421	Hilly/Mountain
Debre Markos - Gondar	Debre Markos-Bure	87.8	18	128	Flat/Rolling
		24.8	55	336	Hilly/Mountain
	Bure-Bahir Dar	156.4	16	113	Flat/Rolling*
	Bahir Dar-Werota	59.3	13	75	Flat/Rolling
		2.4	50	300	Hilly/Mountain
	Werota-Azezo	65.7	16	113	Flat/Rolling
		32.3	63	385	Hilly /Mountain
	Azezo-Gondar	8.8	16	113	Flat/Rolling
		4.4	63	385	Hilly /Mountain
Awash - Kulubi - Dire Dawa - Harar	Awash-Arbereketi	78.1	19	135	Flat/Rolling
		27.5	58	354	Hilly/Mountain
	Arbereketi-Kulubi	12.7	18	130	Flat/Rolling
		114.2	72	444	Hilly/Mountain
	Kulubi-Dengego	7.8	15	98	Flat/Rolling
		23.2	75	465	Hilly/Mountain
	Harar-Dengego	30.0	17	123	Flat/Rolling*
	Dengego-Dire Dawa	11.0	12	71	Flat/Rolling
		8.6	80	500	Hilly/Mountain

* No hilly/mountainous sections

Example: Taking a road section of Woldiya-Maychew with an average degree of curvature 422 degree/km and assume 4 individual curves in a kilometer with L = 200 m; then $\Delta = 422/4 = 105$ and radius of the curve become, R = 110 m which is not recommended for railway line. The result indicates that on significant lines of geometrically constrained, it is necessary to recommend small value of horizontal curves.

3.2. Climatic Variation

Latitude, altitude, winds and humidity with varying magnitude have significant impact on temperature conditions in Ethiopia. Although there are considerable differences between Highland and Lowland Ethiopia in the average monthly and annual temperatures, they are more or less similar in their small annuals range and large daily range of temperatures.

In Ethiopia, temperature is greatly influenced by changing altitude. Lateral variation of relatively few kilometres may result in vertical changes of 1000 meters or more in some of the major canyons, on the slopes of high mountains and along the rift valley escarpments.

As a result Ethiopia has a diversified climate ranging from semi-arid desert type in the lowlands to humid and warm (temperate) type in the southwest. Mean annual temperature ranges from < 15 ^{o}C over the highlands to > 25 ^{o}C in the lowlands; and the mean annual rainfall distribution ranges a maxima (>2000 mm) over the Southwestern highlands and minima (<300 mm) over the Southeastern & Northeastern lowlands [National Meteorological Services Agency, 2001].

In addition, the extreme temperature variations in certain interval of time with specific areas are an important parameter in the design of rail.

Table 3.2: Recorded extreme temperatures in Ethiopia [Temperature Records, 2007]

Place	Extreme Temperature (°C)		Temperature Variation (°C)
	Maximum	Minimum	
Addis Ababa Airport	36	-2	38
Addis Ababa City	34	-3.5	37.5
Dire Dawa	42	5	37
Jimma	38	-8	46

3.3. Availability of Construction Material

The availability of construction material for railway system construction can be roughly determined from the material investigation of Ethiopia Road Authority (ERA) for different road projects; and material investigation study of Ethiopia Railway Corporation (ERC) for the new planned railway systems.

A. ERA Construction Material Investigation

The main requirement properties of construction material to use for Ethiopia road according to Ethiopia Road Authority are:

Borrow material

The main requirement for borrow material to be used for embankment is to satisfy the following requirements:

- Liquid Limit ≤ 55%
- Plasticity Index ≤ 25%
- CBR (after 4 days soaking) ≥ 5%
- Swell on CBR specimen ≤ 2%

With this regard, the design engineer has located potential borrowed material sources at economical hauling distance. The availability of these sources has been checked during the site investigation period. The most common type of borrow material used in different Ethiopia road projects are: yellowish slightly to highly weathered rock; reddish sandy clay; yellowish silty clay; reddish gray silty clay; light brown and brown silty clay; red and black cinder; white gravel with soil; black stone with soil; gravelly sand with few clay; silty clay; partially weathered to decomposed lime-stone and mudstone; and etc.

Sub Base and Fill Material

Representative samples collected from Natural Sub Base / fill pits are subjected to the following tests:

- Particle size distribution
- Atterberg Limits, LL---Max 60, PI---Max 12
- Maximum Dry Density (MDD)
- Optimum Moisture Content (OMC)
- 4 days soaked CBR-------Min 30%
- Percentage Swell---------Max 2%

Note that as per ERA Technical Specification 2002, the requirements for natural gravel subbase is:

- Minimum CBR value of 30 at 95% Modified AASHTO compaction
- Plastic Index (PI) maximum of 6 to 12
- Minimum Grading Modulus (GM) of 1.2
- Plastic Product (PP) not exceeding 75%

The most commonly used type of subbase material in Ethiopia road projects are: slightly to highly weathered gravel (sand stone); highly weathered gravel with yellowish silty clay or with reddish to brown silty clay soil; slightly to highly weathered and decomposed rock; gravelly soil; red and black cinder; pinkish to reddish yellow weathered gravel with silty sand; light yellowish to grey silty gravel; dark silty sandy gravel; grayish to brown sandy silty gravel; partially weathered and fractured basalt; granular materials; moderately weathered to decomposed DOLORITE; and partially weathered and strongly fractured intercalated and amphibole SCHIST.

Quarry Materials

Representative rock samples were collected from quarries and were subjected to the following tests:

- Specific gravity --------------------Min 2.4
- Aggregate Crushing Value------Max 25%
- Los Angeles Abrasion-----------Max 30%
- Flakiness Index--------------------Max 30%
- Sodium Sulphate Soundness Loss----Max 15%
- Coating and Stripping-----------Min 95%
- Water Absorption-----------------Max 2%

The most commonly used type of material for base course in Ethiopia road projects are: fresh slightly weathered basaltic rock; fresh welded tuff/ignimbrite; crushed aggregate; boulder aphantic basalt; trachyte; trachy basalt; Limestone; and sandstone.

Table 3.3: The offset length of available construction material in different road project

Road Project	Offset length (m)					
	Quarry		Subbase		Borrow Site	
	Mean	Standard Deviation	Mean	Standard Deviation	Mean	Standard Deviation
Nekemt-Bedele Upgrading	505.6	1147.2	800	1366.7	125	210
Nehile-AbaAla	315	463.7	0	0	0	0
Woreta-Woldiya	1200	793.7	1082.3	2402.3	517.4	584.6
Kombolcha-Bati-Mille	990.4	1295	84.4	102.6	-	-
Afdera-Abala	1025.7	783.8	1033.2	818.5	2501.5	6534.9
Mekele-AbiAdi – Adwa	335.7	401.8	358	456.4	824	1919
Awash-Dofen	8940	10055	-	-	157.8	176.7

B. ERC construction material investigation

The material investigations carried out along the Addis Ababa – Dewele railway corridor by Addis Ababa university; it has been found that suitable quarry stone sources (mainly basalt and to some extent limestone from new and existing quarry sites) for ballast and sub-ballast are:

- Sufficiently available between Addis Ababa and Meiso,
- Fairly available between Meiso and Dire Dawa but at a relatively great haul distance, and
- Moderately scarce between Dire Dawa and Dewele junctions.

4. Discussion on Different Standards

The discussion on track geometry elements and track structural elements are according to different country standards and different literatures such as doctoral and master thesis, reports, and researches presented to different companies.

4.1. Discussion on Track Geometry Elements

The track geometry element provisions of different standards are compared based on limiting values; the exceptional or permission values are not considered in this discussion for uniformity since exceptional value are allowed by the design engineer at difficult condition.

Track Cant

There are at least two reasons for limiting track cant:

- A very high track cant leads to high lateral accelerations parallel to the wagon floor, if the train is running very slowly and as a result risk of load displacement.
- A high cant leads to formation of high cant excess for slow freight trains and in turn leads to damage and wear of low rail.

For passenger trains a higher track cant can be allowed, because there is no risk of danger if a passenger train stops at a section with high cant. But in exposed places, where high winds may be experienced, it is undesirable to apply cant to the maximum value.

AREMA and German standards provide a maximum recommended limiting value of 100 mm whereas others provide around up to 160 mm. In the new practice and proposed European standards a quite high cant is allowed also on lines with mixed traffic normally limited to160 mm. For speed more than 200 km/hr a maximum value of 110 mm is recommended. Cant on sharp curves must not exceed (R-50)/1.5 (where R is the radius in meters) as indicated by Railway group Standards. A maximum recommended limit of cant, most standards agreement, is 180 mm with a tolerance of 20 mm for maintenance purpose.

Cant Excess

CEN recommended a high value of 110 mm (maximum limiting of 130 mm) cant excess. Whereas other standards recommended a value up to 100 mm and typical limit values of just 50 - 70 mm for slowly running freight trains, for example, in Germany and Sweden. This, in turn, required low cant to be arranged on high-speed lines with mixed traffic and slowly running freight trains.

Cant Deficiency

Similar trends are obvious also for the maximum allowed cant deficiency (lateral acceleration in the track plane). Germany recommended a cant deficiency of 70 mm with a limiting value of 130 mm whereas most others, France, Sweden (Train-B) and TSI (passenger) recommended a limiting value of 150 mm. In addition CEN recommended a limiting value of 160 mm for speed limit of 160 km/hr and a limiting cant deficiency value is decreased as speed increase. For freight traffic lines a minimum value is recommended than passenger lines. Whereas for conventional trains on lines specially built for high-speed, a limit value of 100 mm is recommended. This indicates that as speed increase, it is recommended to reduce cant deficiency.

Horizontal Curve Radius

As discussed from the literature review, provision of design horizontal radius by different standards has similar fashion with the main difference in the provision of equilibrium cant. Even if all standards use similar formula to calculate horizontal curve radius, the value varies to some extent due to variation of track cant and cant deficiency. But for new lines, Sweden recommended that the dimensional speed should be multiplied by a factor, $\gamma = 1.3$ for a purpose of ride comfort and increased speed in the future on the following general equation.

$$R = 11.8 * \frac{{V_{max}}^2}{h_t + h_d} = 11.8 * \frac{{V_{max}}^2}{h_{eq}}$$

Table 4.1: Comparison of horizontal curve radius [m] for different standard

Speed (km/hr)	Standards					
	Sweden		Germany		TSI/CEN	
	$R_{rec,min}$ ($\gamma = 1.3$)	R_{min}	$R_{rec,min}$	R_{min}	$R_{rec,min}$	R_{min}
	h_{eq} =250	h_{eq} =250	h_{eq} = 170	h_{eq} =290	h_{eq} =310	h_{eq} =360
80	520	310	450	260	240	210
100	800	480	700	410	370	330
120	1150	680	1000	590	540	480
140	1570	930	1360	800	730	650
160	2050	1210	1780	1050	950	840
180	2590	1530	2250	1320	1280	1200
200	3200	1890	2780	1630	1580	1480
220	3870	2290	3360	1970	1910	1790

Where: $R_{rec,min}$ is recommended minimum radius and R_{min} is the minimum radius [m].

Transition Curve and Superelevation Ramp

All standards assume a linear variation of curvature (clothoids) and uniform variation of cant through the transition length and the transition curvature shall coincide with superelevation ramps in both shape and position for the purpose of simplicity. Germany, Sweden, Railway Group Standards (RGS) and TSI/CEN have the same permissible limiting values of the rate of cant as a function of length is 2.5 mm/m or a cant gradient of 1 to 400; which is the steepest permitted designed cant gradient through the transition curve length. This indicates that the permissible minimum limiting transition length is ($0.4 * \Delta h_t$).

Table 4.2: Comparison of minimum limiting transition curve length in [m] for mixed traffic lines with passenger trains using maximum cant deficiency difference

Speed (km/hr)	Sweden (Train-A)	Germany	TSI/CEN
	$L_{t,min} \geq \frac{6 * V_{dim} * \Delta h_d}{1000}$	$L_{t,min} \geq \frac{4 * V_{dim} * \Delta h_d}{1000}$	$L_{t,min} \geq \frac{V_{dim} * \Delta h_d}{3.6 * \left(\frac{dh_d}{dt}\right)_{lim}}$
	$\Delta h_d = 150$ mm	$\Delta h_d = 130$ mm	$\Delta h_d = 160$ mm
80	72	42	40
100	90	52	49
120	108	62	59
140	126	73	69
160	144	83	79
180	162	94	89
200	180	104	99
220	198	114	109

Note: - The comparison is based on cant deficiency

- Δh_d is the maximum limiting value and for lower than such values the minimum transition length is less than to the value computed in the table.
- $\left(\frac{dh_d}{dt}\right)_{lim} = 90$ mm/s (maximum limiting value) and transition curve length increase as a rate of cant and rate of cant deficiency decreases.

As the table above shows Germany and TSI/CEN minimum recommendations are almost similar whereas the Sweden is much greater than them. In computation of transition curve lengths, Sweden used constants of for train type and Germany also used constants whereas CEN provisional standard incorporate rate of cant deficiency in addition to dimensional speed and variation of cant deficiency.

Gradient

For all standards on tracks with only passenger trains and light freight trains higher values in excess of 1% can be allowed. The largest permissible gradient of Sweden is 1% on track with heavy freight trains and 1.25% if the mean value does not exceed 1.0% over each kilometer. Germany railway largest permissible gradient is 1.25% for mixed traffic main lines and provides high gradient of steeper than 4% for commuter line; and TSI also provides high value of gradient, typically up to 3.5%, by limiting the length of continuous gradient.

Vertical Curve Radius

The minimum requirements are quite similar to each other, including Sweden standard to some extent. Technical Specification of Interoperability (TSI) recommended radius with the limited value of less than or equal to 230 km/hr speed; whereas others recommendations discussed are for high speed lines. Normally, the required vertical curve radius is somewhat larger on crests than in hallows. This is due to the risk of wheel unloading on crests. Vertical alignment of stabling and service tracks, agreement of all, shall not include curves of radii less than 600 m on a crest or 900 m in a hollow.

Table 4.3: Comparison of vertical curve radius [m] for different standard

Speed (km/hr)	Standards					
	Sweden		Germany		TSI/CEN	
	$R_{rec,min}$	R_{min}	$R_{rec,min}$	R_{min}	$R_{rec,min}$	R_{min}
80	2700	1100	2600	1600	2300	1600
100	4300	1600	4000	2500	3600	2500
120	6100	2300	5800	3600	5100	3600
140	8300	3200	7900	4900	6900	4900
160	10900	4100	10300	6400	9000	6400
180	13700	5200	13000	8100	11400	8100
200	16900	6400	16000	10000	14100	10000
220	20500	7800	19400	12100	17000	12100

Where: $R_{rec,min}$ =recommemded minimum vertical curve radius and R_{min} = minimum vertical curve radius. The values have been rounded up to nearest 100 m.

4.2. Discussion on Railway Track Structure

4.2.1. Rail

Even if the rail sections are connected in the field by either bolted joints or welding; on today's high performance lines, according to Pen (2008) welding is better to create continuously welded rails (CWR) to eliminate a potential cause of dynamic load. In addition bolted joints are sensitive to excessive abrasion wear at joints and the impact load on joints also causes pad deterioration, fastener looseness and geometry deterioration.

Rails of a shape in section somewhat like an I-beam are used now days in all standards to provide flexural stiffness and strength; and the most commonly used rail profile is flat-bottom rail, also called Vignole rail. The mass per metre of a rail can be computed, practically, based on the design load and speed of the track.

Dynamic Impact Factor ($\emptyset$)

Dynamic impact factor, a corrective factor to compensate for dynamic as well as impact effects of wheel load resulted from wheel and rail surface irregularities, is determined empirically and is always expressed in terms of train speed in all standards (Doyle, 1980; Sadeghi and Barati, 2010).

Table 4.4: Comparison of dynamic impact factor (∅) of different country standards [Doyle, 1980; AREMA; 2009 and Sadeghi and Barati, 2010]

Speed, km/hr	80	100	120	140	160	180	200	220
AREMA $(1+5.21\frac{V}{D})$	1.46	1.58	1.69	1.81	1.93	2.04	2.16	2.27
German (DB) $(1+\frac{4.5*V^2}{10^5}-\frac{1.5*V^3}{10^7})$	1.21	1.33	1.39	1.47	1.54	1.58	1.6	1.58
India $(1+\frac{V}{58.14k^{0.5}})$	1.21	1.27	1.32	1.37	1.43	1.48	1.53	1.59

Note: - For German standard, $\emptyset = 1+\frac{V^2}{3*10^4}$ for V≤100 km/hr
- Track modulus (k) of concrete tie track is approximately 6ksi (41.4Mpa)
- Wheel diameter (D) of AREMA method is assumed 900mm for this study.

The AREMA dynamic impact factor value is greater than that of the other since the country use high axle load vehicles in the world. As a result AREMA rail grade specification has higher tensile strength than European provision in general.

To select appropriate rail grade according to allowable bending stress value, bending stress value of these standards are tabulated below with the following assumptions.

Steel rail elastic modulus, E = 207Gpa (30×10^6Psi)

Track modulus, k = 6000 lb/in^2 or 41.4 Mpa (concrete sleeper)

An axle load of 18 ton (18×10 KN), then wheel load- $P_s = 90$ KN

100RE (101.5 lb/yard or 50kg/m)

Moment of inertia, I = 49 in^4 or 2039 cm^4

Distance to neutral axis, c = 2.75 in or 6.985 cm

Section modulus, Z_b = 17.8 in^3 or 291.7 cm^3

Then

Bending stiffness of a rail, EI = 2039cm^4 × 207Gpa = 4.22×10^6 Nm^2

Design wheel load, $P = \emptyset \times P_s = 90\emptyset$KN

Stiffness ratio of equation (2-44) can be

$$\beta = \sqrt[4]{\frac{k}{4EI}} = \sqrt[4]{\frac{41.4}{4\times4.22}} = 1.25143$$

A designed bending moment is maximum at load point, x = 0. Therefore, substituting β and x to equation (2-43) provides

$$M_{max} = \frac{P\ e^{-\beta x}}{4\beta}(\cos\beta x - \sin\beta x) = \frac{P\ e^{-0}}{4 \times 1.25143}(\cos 0 - \sin 0) = \frac{P}{5.006}$$

The maximum stress from equation (2-45) can be

$$\sigma_{max} = \frac{M_{max} * c}{I} = \frac{M_{max}}{Z_b} = \frac{P}{5.006 \times 297.1 \times 10^{-6}} = 672.4 \times P$$

Assume the effect of eccentricity of vertical load, twisting by flange force and lateral deflection under flange force will increase the rail bending moment and/or bending stress by 50%, then

$$M_{max} = 1.5 \times \frac{P}{5.006} = \frac{P}{3.3373}$$

$$\sigma_{max} = 1.5 \times 672.4 \times P = 1008.6 \times P$$

Table 4.5: Comparison of bending moment and bending stress of rail of different standards for 18 ton axle load on 50kg/m rail [Krishna et. al., 2006 and AREMA, 2009]

Speed, V (km/h)	AREMA		DB		India	
	M_{max} (KN.m)	σ_{max} (Mpa)	M_{max} (KN.m)	σ_{max} (Mpa)	M_{max} (KN.m)	σ_{max} (Mpa)
80	39	133	33	110	33	110
100	43	143	35	118	34	115
120	46	154	37	126	36	120
140	49	164	40	133	37	125
160	52	175	41	140	39	130
180	55	185	43	144	40	134
200	58	196	43	145	41	139
220	61	206	43	143	43	144

The allowable rail bending stresses of different countries commonly depend on tensile strength of the steel rail. But AREMA and DB (Germany railway) allowable rail bending stresses also consider temperature induced stresses [Krishna et. al., 2006]. The allowable rail bending stress ranges of different steel grade rail are tabulated in the Table 4.6.

Table 4.6: Comparison of allowable bending stress (MPa) for different rail grade at 15°C temperature variation [Vitez et al, 2004; Jain and Murthy, 2008; Krishna et. al., 2006 and AREMA, 2009]

UIC Steel Rail grade	Tensile strength (Mpa)	Yield strength (Mpa)	AREMA	DB	India
			σ_{all}	$(\sigma_{all})'$	σ_{all}
R700	680	380	150	164	197
	830	450	185	204	241
R900A/ R900B	880	480	200	220	255
	1030	510	215	237	299
R1100	1080	650	285	316	313

Note: $(\sigma_{all})' = 0.5625(\sigma_y - 1.111\sigma_t)$; $\sigma_{all} = 0.29 \times f_{yk}$- Indian provision; and $\sigma_{all} = \frac{\sigma_y - \sigma_t}{(1+A)(1+B)(1+C)(1+D)}$ – AREMA

- Assume variation of temperature, Δt = 15°C = 59 °F to compute thermal stress
- Thermal stress, $\sigma_t = E \times \alpha \times \Delta t$ = 207Gpa × 6.5×10^-6 °F × 59 °F = 79.4Mpa

– The sever case is assumed for safety factor of AREMA

From the above table AREMA allowable bending stress value is relatively less than the others and this indicates that they underestimate the capacity of the rail to resist dynamic load impact i.e. it is relatively the safest method. As a result AREMA needs higher steel grades than others. For example, for above assumption, taking a speed 180 km/h, according to AREMA the bending stress (185 MPa) is equal to the allowable bending stress (185 MPa) and needs to provide a steel grade of at least R900A (R260) whereas providing steel grade of R700 (R200) for DB and India satisfy the allowable requirements since the bending stress is less than allowable bending stress of R700.

Most railway in Europe, including UIC recommendation, select standard rail grade R260 or UIC 900A for horizontal curve radius of greater than 700 m whereas higher standards are used for radius less than 700m to avoid wear and also rolling contact fatigue (RCF) up to 1500 m curve radius [Vitez et al, 2004 and Pointner, 2010]. AREMA recommends a higher rail grade steel than European standards since their standard rail have a minimum HB of 310 and yield strength of 511 Mpa (AREMA, 2009). Whereas UIC 900A (R260) have a maximum HB of 300 and maximum yield strength 510 [Vitez et al, 2004; Jain and Murthy, 2008; and Pointner et al, 2010]. The rail grades of less than R260 are not recommended anymore for mainline railways [Pointner et al, 2010].

4.2.2. Fastening System

Concrete tie fastening systems are comprised of various components and materials designed to safely transmit forces exerted by the rail to the concrete tie while restraining the rail to the proper gauge and cant (Gutierrez et al, 2007). The selection of fastening system and performance tests are discussed below according to different literatures.

Selection of fastener types

The type of fastening systems may vary in durability, elasticity, ease of installation, ease of maintenance, amount of maintenance required, clamping force, contact area with the rail, cost, design life, and whether or not they provide a vandal-proof design (Gutierrez et al, 2007).

The choice of fastening is greatly depend on the properties and structure of the sleeper. Spikes are commonly used for wood sleeper and elastic fastening used for concrete sleepers in which the rail is fastened by a spring using rubber pads (Miura et al, 1998; Esveld, 2001). In addition, the type of elastic fastening selected in different countries are based on cost of the fastener, ability of fastening force, capacity of bearing ability and easy adjustability of the fastener (Miura et al, 1998).

Performance Test on Fasteners

Performance tests are conducted by both AREMA and European standards for fastening systems to resist the effect of acting forces and environment. The most common tests on fastening systems practiced in Europe and North America are repeated load test, longitudinal load restraint, determination of clamping force, stiffness of rail pad test, electrical resistance (AREMA, 2009; EN 13146-4, 2002). Torsional resistance is done only by European standards in horizontal movement of train. The other differences between these standards are the test parameters used (e.g. applied loads) and method of performing such performance testing.

4.2.3. Sleeper (Crosstie)

Type of sleeper material

The type of sleeper material can be selected according to economical advantages but the properties of each sleeper material also indicate the preferable one. Steel sleepers are most expensive relative to wood and concrete (Esveld, 2001). Concrete sleepers are more widely used than others since it is not affected very much by either climate or weather; it resists track buckling better; and it generally has more secure fastening systems, so concrete holds the rails better and due to such and other advantages for analysis purpose prestressed concrete sleepers (PCS) are considered.

Sleeper spacing and dimension

The range of sleeper spacing of both AREMA and Australia standards are almost similar; AREMA (2009) provides between 510 mm to 760 mm whereas Australia standard (AS1085.14, 2003) provides between 500 mm to 750 mm. As spacing of sleeper increase, load distribution to a single sleeper also increase and as a result it may need a high quality sleeper with high bearing area or reducing axle load. On the other hand as sleeper spacing decrease, high number of sleeper per kilometer track are needed and it may be costly but may be efficient to transfer load from rail to ballast. Furthermore, AREMA recommends sleeper length between 7 ft. 9 inches (236 cm) and 9 ft. (274 cm) with the base width between 8 inches (20 cm) and 13 inches (33 cm) whereas Australia standards recommend to design by the bond development requirements of the prestressing tendons and the base width is then calculated from allowable bearing pressure.

Rail Seat Load and Sleeper-Ballast Contact Pressure

The magnitude of rail seat load ($q_r = DF \times P$) can be computed from the designed wheel load and distribution factor (DF) that depend on sleeper spacing. Both AREMA (2009) and Australia standard (AS1085.14, 2003) have similar general formula but the design wheel load is factored with a factor (dynamic augment for speed and rail irregularities) of 300% ($1+\emptyset$, $\emptyset = 2$ is assumed impact factor) and minimum of 250% respectively. The variation is due to the basis of their experience in past.

In addition, the average pressure at the bottom of the sleeper ($P_a = q_r/A$) of AREMA (2009) and Australia standard (AS1085.14, 2003) vary with computation of rail seat load as discussed above and effective bearing area ($A = b \times L$) of cross sleeper that differ on effective length of sleeper. AREMA and Australia standards recommends the effective length of sleeper as ($L = l/3$) and ($L = l - g$) for standard and broad gauge.

Assuming axle load of 18 tonne (wheel load of 90 kN); the design sleeper spacing of 600 mm; the sleeper average base width of 300mm; the length of sleeper of 2640 mm; and an impact factor of 300%. From Appendix C, Figure C-3; the distribution of load according to AREMA and Australia standard are 50% and 51%. Therefore, the rail seat load and average pressure are summarized in table 4.7.

Table 4.7: Comparison of rail seat load and average sleeper-ballast pressure [AREMA, 2009 and AS1085.14, 2003]

	Rail seat load, q_r (KN)	Average pressure, P_a (KPa)
AREMA	135.0	511.4
Australia standard	137.7	402.6

With the given assumption, the rail seat loads for both standards are almost equal since the difference of load distribution factor is small. But the average sleeper ballast pressure of AREMA is much greater than that of Australia standard computation since the effective length of sleeper in the calculation of bearing area for Australia standard is greater than that of AREMA. This indicates the Australia sleeper pressure spreads in higher area than AREMA and needs continuous maintenance to restrain it. In addition the AREMA value is also much greater than the recommended ballast pressure of 65 psi (450 KPa) since AREMA not only underestimate the effective bearing area between sleeper and ballast but also capacity of ballast to resist pressure than Australia standards. As a result AREMA may need high-quality abrasion resistant ballast or large bearing area of sleepers to reduce the contact pressure between sleeper and ballast. So designs of sleeper dimension both breadth and length according to AREMA become larger than Australia standard, accordingly the cost also increases. AREMA approach is extremely conservative and it is highly costly.

Design Bending Moment of Sleeper Critical Section

The AREMA specification does not provide formulas for calculations of the moments. Instead, it provides graphs for the design moments, with various sleeper lengths and sleeper spacing. This diagram shows an unfactored positive bending moment at rail seat and then it must be multiplied by the speed and tonnage factor. Negative rail seat moment and moment at center of the sleeper can be computed by multiplying unfactored positive bending moment by moment factors related to tie length [AREMA Sec. 4.4.1.2, 2009]. On the other hand Australia standard [AS1085.14, 2003] has a formula to compute bending moment at rail seat and sleeper center considering rail seat load; and in addition it does not consider speed and tonnage factor.

Table 4.8: Comparison of critical sections design moment of PSC for axle load of 18 tonne, sleeper length of 2640 mm and sleeper spacing of 600 mm [AREMA, 2009 and AS1085.14, 2003]

Standard	Rail Seat Moment (kN.m)		Center Moment (kN.m)	
	M_r^+	M_r^-	M_c^+	M_c^-
AREMA	42.35	21.47	18.51	26.38
Australia (AS1085.14)	19.62	14.00	7.85	12.39

Note: The AREMA annual tonnage and train speed are assumed of 75MGT and 100 mph respectively for comparison purpose.

As the table above indicates, the AREMA moment calculation does not depend on actual load of train instead considers annual tonnage factor and moment values that are varying with sleeper spacing and length. Whereas Australia standard uses exact applied design load with sleeper spacing and length; and the moment increases as design load increases.

As a result of over estimation of design moment, AREMA needs higher depth of sleeper with same base plan than Australia provision to reduce bending stress that depend on the moment of inertia of the sleeper. This makes AREMA costlier than Australia but of course they may be safer.

Material Properties

The material properties of concrete and steel play important role not only in design of reinforced concrete structures but also PSC sleeper. As a result the recommended minimum concrete strength (f_{cp}) according to Australia standard and AREMA are 50 Mpa and 48 Mpa respectively (AS1085.14, 2003 and AREMA, 2009). The maximum permissible compressive stress at transfer is $0.55f_{cp}$. In addition the recommended steel for tendon is a minimum strength, for example, s-250 with ultimate strength of 255 ksi and yield strength of 230 ksi.

Design of Sleepers

The current international design standards for prestressed concrete (PSC) sleepers are based on the permissible or allowable stress of materials concept [Remennikov et al, 2008]. But AREMA uses limit state design with the reference of America Concrete Institute-ACI 318 [Lutch, 2009] and in addition the collaborative research between the University of Wollongong (UoW) and Queensland University of Technology (QUT) in Australia develops limit states design concept [Remennikov et al, 2008]. According to permissible or allowable stress of materials concept, flexural failure in ties are defined by steel rupture or concrete crushing; whereas according to limit states, failure is defined by the propagation of cracks in the sleeper [Krishna et al, 2006 and Lutch, 2009].

According to limit state design approach, the applied stresses due to prestressing and applied load (i.e. stresses at transfer and stresses at service), shall be less than allowable concrete stresses [Lutch, 2009]. Losses are included in computation of effective stresses and Australia standards recommend a specific value of prestress losses whereas according to Naaman (2004) cited by Lutch (2008) prestress losses can be calculated theoretically on ACI consideration. As a result ACI prestress loss calculation needs accuracy of variable assumptions and Australia assumption overestimate prestress losses to avoid failure.

Finally the cracking moment according to both AREMA and the newly developed Australia limit state design are checked both theoretically and experimentally [Remennikov et al, 2008 and AREMA, 2009].

4.2.4. Track Ballast

Ballast Material

The ability of ballast to perform its functions is controlled by the particle characteristics together with the physical state of the assembly (grain structure and porosity). Different researchers agreed on that 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 [Bonnett, 2005; Esveld, 2001; Pen, 2008; Indraratna et al, 2006]. A wide variety of materials are used throughout the world such as basalt, limestone, granite, dolomite, quartzite, gravel, rheolite, slag, etc.

Ballast Material Specification

Even if railway engineers are mainly interested in mechanical and dimensional properties, the railway authorities worldwide did not agree on which are the optimum ballast index characteristics (gradation, grains size, shape, hardness, abrasion resistance and mineral composition) that will provide the best track performance [Aursudkij, 2007; Ionescu, 2004].

Each railway system (standard) performed a complement test which satisfies the characteristics of the specification that should be needed. Even if they use similar tests, the limited values may be different since there is no common agreement on characterization.

The AREMA durability tests are completely different from other standards; but degradation test is equivalent to LAA, MDA and WAV tests since each test involves measuring the degradation of ballast in a revolving drum [AREMA, 2009; McDowell, 2006].

In addition according to Table 4.9, shape tests of all standards are almost similar for all standards except one additional property of Australia standard. Flakiness and flatness; and elongated particle and elongation index are different names for the same shape characteristics in different standard definition [Selig and Waters, 1994]. The ratio of thickness to width and length to width of a particle for each standard may be different. These properties are defined in terms of percent of mass in a sample and shape index of Finland is related to flatness and elongation characteristics of the particle. In addition the ERA manual performs durability and shape characteristics tests for crushed rock aggregate of base course even if the limiting value is vary from railway provision.

The environmental impact test and sulphate soundness on railway ballast are done only in US (AREMA, 2009). But in Ethiopia road construction it is also practiced to check whether the material is subjected to weathering action or not (ERA, 2002).

Table 4.9: Comparison of ballast specification properties of different railway standards [AREMA, 2009; ETA-04-01 Ver1.1, 2007; Consta JV, 2007; and EN 13450, 2002 cited by Nurmikolu (2005)]

Tests	Railways					ERA
	US	UK	Australia	Finland	Ethio-Djibouti	
Durability	- Degradation - Clay Lumps & Friable Particle	- LAA - MDA	- WAV - LAA - ACV	- LAA - MDA - Nordic ball mill	- LAA - AIV - Crushing resistance	- LAA - AIV
Shape Tests	- Flatness & elongated particle	- FI - PLI	- FI - Misshapen particle - uncrushed aggregate	- Shape index	- FI - EI	- FI
Environmental test	- Sulphate Soundness	-	-	-	-	- Sulphate Soundness
Stability	- Specific gravity	- Specific gravity	- Specific gravity	- Specific gravity	- Specific gravity	- Specific gravity

Table 4.10: Comparison of different railway standards based on different criteria [AREMA, 2009; ETA-04-01 Ver1.1, 2007; Consta JV, 2007; and EN 13450, 2002 cited by Nurmikolu (2005)]

Criteria	US		UK	Australia	Finland	Ethio-Djibouti
	AREMA 25	AREMA 4A				
Sieve size (mm) corresponding to 100% pass	75	63	63	63	80	70
Sieve size range for high mass proportion (mm)	"d" /63	19/50	31.5/50	26.5/53	31.5/63	32/64
Sieve size limits the fines content to 1%	0.075	0.075	-	0.075	0.063	0.075
Coefficient of uniformity (C_u)	≥ 3	< 2	< 2	< 2	< 2	< 2

Note: "d" is between ½ inch (12.7 mm) and No.4 (4.75 mm) sieve size, most likely 9.5 mm.

All standards agree on that the fines particle pass on sieve size No.200 (0.075 mm) or 0.063 mm (for Finland) is limited to 1%. In addition most standards also have coefficient of uniformity less than 2 even if the reason for the choice of these gradations are not clearly explained. But based on Indraratna et al (2004) cyclic test findings, cited by Indraratna and Salim (2005), a coefficient of uniformity between 2.2 and 2.6 is recommended for ballast gradation.

All AREMA gradations of lower gradation boundary is lower than that of other standards even except AREMA 3 gradation the lower boundary is less than 22.4 mm which is the grain size diameter for the permissible percentage of fines as Wenty (2007) indicated. Whereas the other standards and Ethio-Djibouti railway lower gradation boundary limits are greater than 22.4 mm.

According to the gradation of different standards, the AREMA 25 [AREMA, 2009] is broadly graded (well-graded) than other gradations, even C_u can be greater than 4. Therefore, this gradation has good shear strength and as a result the track will be relatively stable and minimum track deformation, but suffer from drainage problem in long term. On the other hand the standard that uses uniform graded ballast; UK, Australia, Finland and Ethio-Djibouti railway, can attain high permeable track ballast but may suffer from degradation, settlement and track instability according to Indraratna and Salim (2005) extensive research. As a result the optimum ballast gradation needs a balance between the uniform and broad gradation for a purpose of drainage and strength respectively.

Ballast section depth

The absolute minimum depth of ballast needed beneath sleepers for even a lightly loaded railway should never be less than 150mm. The recommended limit of ballast depth according to AREMA and RGS (for concrete sleeper) is 300 mm, and Australia and UK have a minimum recommended value of 200 and 225 respectively [AREMA, 2009; ETA-04-01 Ver1.1, 2007; GC/RT5014, 1995; and EN 13450, 2002 cited by Krishna (2006)]. The required depth of good quality ballast beneath sleepers varies depending upon the maximum speed of trains, the maximum axle loads carried and the gross annual tonnage expected.

4.2.5. Track Subballast

Similar to ballast, subballast is also a granular material but is generally finer and more broadly-graded than ballast to resist permanent deformations at lower stress levels than prevail in the ballast layer, ignoring the requirements on tamping efficiency [Nurmikolu, 2005; Aursudkij, 2007]. In addition this broadly graded subballast must fulfill the filter requirements for the ballast and the sub-grade [Lim, 2004]. Selig (2004) indicates that the durability requirements are not as severe as for ballast because the subballast particles are smaller and the stresses are lower.

Subballast Material

Material most commonly used as sub-ballast include crushed stone, natural or crushed gravels, natural or crushed sands or a mixture of these materials [AREMA, 2009; FRA, 2002 cited by Nurmikolu (2005)]. From the material selection point of view, the materials identified for ballast can be used as good sub-ballast material.

Subballast material Specification

Researchers [Selig, 2004; Boras, 2004; Nurmikolu, 2005] agreed on that sub-ballast materials should be broadly-graded sand-gravel mixtures to enable good compatibility, which must fulfill the filter requirements for the ballast and the sub-grade.

Gradation of Subballast material

As discussed in ballast material specification section, the railway authorities worldwide did not agree on characteristics materials that will provide the best track performance [Aursudkij, 2007; Ionescu, 2004].

The grain size distribution of the sub-ballast according to AREMA (2009) can be determined by applying the filter principle used in drainage to the grading of the subgrade material. Whereas the recommendation of FRA cited by Nurmikolu (2005) and the suggestion of Consta JV (2007) for Ethio-Djibouti railway lines for gradation of crushed rock aggregate differ from AREMA. The AREMA way of gradation determination is more formal than others even the EN standard (EN 13242) does not naturally set requirements for grain size distribution.

The suggestion of Consta JV, directly taken from Spanish provision, for Ethio-Djibouti railway lines for subballast gradation requirements of aggregate (crushed rock, gravel and sand) are slightly finer than the aggregates ordinarily specified and used in construction of Ethiopian roads base course material with nominal size of 37.5mm [Consta JV,2007; ERA, 2002].

According to Selig (2004) depending on the permeability requirements for drainage, the fine particles must not exceed 5 to 10 percent by weight and the recommended limit of fines material passing on sieve 0.075 mm or 0.063 mm for US is 5%, for Finland is 3% and for Ethio-Djibouti railway line is 3-9%. Selig (2004) also added that the finest subballast particles shall be smaller than the largest subgrade particles, and correspondingly the largest subballast particles must be larger than smallest ballast particles.

Test for Subballast material

The Finland subballast test requirements include durability tests such as impact and ball mill; the durability consideration may be severe since the subballast particles recommended are larger that may approach to ballast material size and the stresses will not be low. On the other hand AREMA performs LAA (durability test), Atterberg limit, moisture-density relation, Sulphate soundness and permeability tests. A test that suggested by Consta JV (2007) for Ethio-Djibouti railway to be performed are organic and sulphate content, water absorption, LAA test, CBR test and permeability.

A common test LAA, generally, a higher value is acceptable for subballast than for ballast because the traffic stresses are lower and the smaller size and broader gradation makes the particle contact stresses much lower than in the ballast [Selig and Waters, 1994].

Subballast section depth

The minimum depth of subballast needed beneath the ballast should never be less than 150mm. However, there is wide variation in the practices followed in different countries of the world; almost all leading world railways provide a layer of sub-ballast along with ballast. The recommended limit of ballast depth according to AREMA is 150 mm, and Australia is 150 [AREMA, 2009; Krishna (2006)]. Since the distribution of loads to depth is approximately the same regardless of the granular material, the combined depth of sub-ballast and ballast is calculated as a single unit to develop the pressure on the subgrade.

4.2.6. Subgrade Soil

The railway subgrade, a deep layer either compacted natural ground or an imported fill embankment, must have sufficient bearing capacity, provide good drainage and yield a tolerably smooth settlement in order to prolong track serviceability under specific operating and climatic conditions [Radampola, 2006; Selig, 2004; Kaewunruen and Remennikov, 2008; Piotrowski, 2007].

The important subgrade failure identified by Radampola (2006) usually occurs due to over stressed conditions, poor construction or maintenance practices. Similarly, natural conditions such as weak subgrade soil (silt and clay), high ground water tables and erosion or sliding of embankment also affect the performance of the subgrade.

Subgrade Soil Index Property Tests

The index property tests most commonly selected to characterize subgrade soils by different researchers [Hagos Gebretsadik, 2006; Piotrowski, 2007; Nadew Abdisa, 2010; Zelalem Worku, 2010] are grain size distribution tests (sieve analysis and hydrometer test), compaction (moisture-density relation test), Atterberg limit tests, and CBR (strength property) tests. In addition specific gravity test is recommended by AREMA (2009).

Subgrade Soil Strength

The most common methods of characterizing subgrade soil strength is CBR test and is largely related to their compaction density, which depends significantly on the particle size distribution [ERA, 2002; Radampola, 2006; Veisi et al., 2010].

Subgrade Soil Stabilization

Stabilization of subgrade soil is usually adopted with the purpose of rendering plastic soils coherent to the standards and requirements of engineering projects either by mechanically or chemical additives [Piotrowski, 2007; Benedetto, 2010; Veisi et al., 2010]. The effect of stabilization is quantified by a significant increase in the Atterberg Plastic Limit and by a relevant reduction of plasticity (Plastic Index) [Benedetto, 2010].

Many researches indicated that the selection of stabilizer mainly depend on the property of soils to be stabilized (plastic index and grain size distribution) and the desired engineering properties [ERA, 2002; Little and Nair, 2009; Veisi et al., 2010].

Different types of stabilization methods are used now a day in the world including Ethiopia [Piotrowski, 2007; Veisi et al., 2010; Alemayehu Ayele, 2010]. Geosynthetics mainly used as reinforcement (a class of geomaterials) including geotextiles, geogrids, geocomposites, and geomembranes, which is a method of mechanical stabilization that is gaining popularity throughout the commonwealth, may be expensive [SUDAS, 2010].

Vertical pressure on the subgrade layer

Talbot's equation is empirical whereas Boussinesq elastic theory and Load spread method are both based on the stress in an elastic body due to an applied surface point load. In addition Talbot's equation was developed from a number of full scale laboratory tests. Several types of ballast were tested, including sand, crushed stone and gravel with pressures from applied loads measured at various depths and locations under several sleepers [Selig and Waters, 1994].

4.3. Discussion on Railway Safety System

The railroads are required to keep their operations safe and put up on the railway infrastructure, vehicles and equipment safely and keep in a safe operating condition. Therefore, rail facilities and vehicles must be such that they satisfy the requirements of security and regulation [Braband 2007].

The factors which causes train accidents in railway transportation systems are mechanical problems (signal failures, equipment failures and train control), human factors, infrastructure defect (track, roadbed and structure), and environment or external factors (weather-related, obstruction). The most commonly type of accident occurred in the systems are derailment and collision (especially at grade crossing). In addition fire and violent eruption (including sabotage or terrorism) are also other type of accidents in railway transport system.

Railway safety assessments related to these factors, which are the causes of train accident, are done in railway companies to alleviate the corresponding problems (accidents) occurred in the track system (Zhao et al, 2006; Noh et al., 2010; European Railway Agency, 2010). Railway safety assessment may be classified as safety management system (SMS) and engineering safety management (ESM) documents (Noh et al., 2010).

The purpose of a SMS is to provide a systematic way to control risk and to provide assurance that the system is effective and provides for a high level of safety [Noh et al., 2010]. SMS leads to an enhanced safety culture – it is a journey requiring cultural change on the part of the railways and the regulators (Bourdon 2005). The documents of SMS and ESM are indicated in Appendix D, Part-3.

4.4. Discussion on Platform Dimensions

The Chinese code of standards provide a clear recommended value for each level platform whereas the AREMA provides for the low and high level platform and railway group standards (RGS) recommends single value for all platform system. But the recommended values of Chinese code is more or less larger than that of others, of course the railway group standards recommendation are for all type of platform.

Table 4. 11: Platform dimensions of different code of standards for standard gauge track

Standard		Height of Platform edge from top of rail (mm)			Offset distance from adjacent track (mm)		
		Low level	Ordinary	High level	Low level	Ordinary	High level
China	Passenger	300	500	1250	1000	1000	1000
	Freight	-	1100	≤4800	-	1000	1100
AREMA (Passenger)		210	-	1310	930	-	950
RGS		915			750		

5. Organization of different Track Infrastructure Elements

5.1. Track Geometry Elements

Track Cant

Cant is provided in order to reduce or eliminate lateral acceleration created due to small curve, especially in parts of Ethiopia constrained by difficult topography, when it is not possible to make a suitable large curve radius. Higher values of cant than 160 mm should not normally be chosen when mixed traffic with freight trains would come into question. This is mainly due to the risk of loads "moving around" on the floor of the wagon. Therefore maximum cant value of 180 mm can be set with normal limiting value of 160mm and most commonly a recommended limit of 150 mm. In addition as speed of fastest train increase, cant excess should be too high for slow trains and as a result to avoid high cant excess track cant shall be reduced. This can be achieved by increasing the value of cant deficiency.

The possible recommended value of cant is indicated in Table 5-1 and the restricted value of track cant for sharp curve shall be:

$$h_{t,max} \leq (R - 50) / 1.5 \qquad (2\text{-}22)$$

Where: $h_{t,max}$ = cant in mm and R = 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 5.1: Recommended and maximum limiting value of track cant

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.

Cant Deficiency

When lines are mixed traffic, 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. As speed increases, to reduce the impact of fast train on rail, track cant can be increased and permitted value of cant deficiency become reduced; and as a result for slow traffic the cant excess are increased. So selecting the cant deficiency is an economic balance between the low rail damage caused by the heavy freights and the high side gage face wear caused by the passenger (fast) trains. An increase in cant deficiency leads to faster deterioration of the track geometry. In addition according to comfort criteria, the non compensated lateral acceleration should not exceed 0.1g for non tilting trains. In exceptional condition at very strict topographical constraints, according to the judgment of the engineer, the lateral acceleration may be taken up to 0.15g. Therefore, the maximum limiting value of cant deficiency according to equation (2-15) shall be:

$$h_d = \frac{S}{g} * a_y = \frac{1500\ \text{mm}}{g} * 0.1g = 150\ \text{mm}$$

Table 5.2: Recommended and maximum limiting value of cant deficiency

Speed (km/hr)	Cant Deficiency, h_d (mm)	
	Recommended limiting Value	Maximum limiting Value
$200 \leq V \leq 300$	100	150
$160 \leq V < 200$	140	150
< 160	150	150

NB: For lines whose construction involves very strict topographical constraints especially in Northern part of Ethiopia, an exceptional higher value can be provided. ‘

Cant Excess

Low value of cant excess is recommended to avoid serious damage of low rail due to heavy freights loads since damage of rail is proportional to the weight that loaded on it. As discussed in literature review a recommended value of about 50-70 mm and lower cant excess can be provided for slow speed train on mixed traffic lines. In addition a 110 mm maximum limit value can be provided. For slow moving trains, a minimum lateral acceleration is provided to avoid excessive damage.

Horizontal Curve Radius

Some of track geometry design parameters are believed to have significant influence in the average construction cost of newly built railways, namely horizontal curve radius, vertical curve radius and gradient. As a result appropriate provisions of such elements are necessary.

Radius of horizontal curve for mixed traffic line can be optimized using the following two equations (equation 2-23 and 2-24), from allowable curve radius for the maximum and minimum operating speed, by compromising cant deficiency and cant excess.

$$R_{max} = 11.8 * \frac{{V_{max}}^2}{h_t + h_d} \qquad (2-23)$$

$$R_{max} = 11.8 * \frac{{V_{min}}^2}{h_t - h_e} \qquad (2-24)$$

Equating these two equations to determine the optimized cant for the two phenomena (cant excess and cant deficiency) provides:

$$h_t = \frac{h_e \left(\frac{V_{max}}{V_{min}}\right)^2 + h_d}{\left(\frac{V_{max}}{V_{min}}\right)^2 - 1} \qquad (5-1)$$

In the following table, Table 5.3, the optimized cant, h_t [mm] and optimized radius R [m] are determined from various speeds [km/hr], cant deficiency, h_d [mm] and cant excess, h_e [mm]. In addition the optimized cant and optimized horizontal curve radius, Table 5-3, is summarized from Appendix D of Table D-1 with the recommended limit of cant deficiency and cant excess.

Table 5.3: Optimized horizontal curve radius and optimized cant for mixed traffic lines for different value of cant deficiency and cant excess

V_{max} (km/hr)	V_{min} (km/hr)	h_d (mm)	h_e (mm)	h_t(optimized) (mm)	R_{min} (optimized) (m)
140	60	90	65	100	1218
	80		40	103	1198
160	60	110	65	99	1442
	80		50	103	1416
	100		25	112	1364
180	60	110	70	93	1888
	80		55	96	1859
	100		30	93	1888
200	80	100	65	98	2266
	100		50	103	2213
	120		25	101	2238

NB:- For different assumed value of cant deficiency and cant excess, different optimized value of track cant and horizontal curve radius can be obtained.

- For specified line of dedicated speed, horizontal curve radius can be computed considering the limited value of cant and cant deficiency using equation (2-23).
- The minimum radius shall not be less than 200 m and 150 m on passenger and non-passenger running lines respectively. The exceptional lower value may be provided according to the decision of the design engineer for topographically constrained lines.
- Therefore, the degree of curvature at any condition, the degree which encircled by 100 ft curve, shall not greater than 12 degree from equation (2-2).

Possible example of horizontal curve radius are summarized in the following tables for design speed of new built up lines at different values of track cant [mm], cant deficiency [mm] and speeds that varied from 140 km/h to 260 km/h.

Table 5.4: Horizontal curve radius [m] at four different values of track cant (Assume Cant Deficiency, $\mathbf{h_d}$= 80 mm)

Cant, h_t (mm)	Design Speed, V (km/hr)						
	140	160	180	200	220	240	260
100	1285	1678	2124	2622	3173	3776	4432
130	1101	1438	1821	2248	2720	3237	3798
150	1006	1313	1662	2052	2483	2955	3468
180	890	1162	1470	1815	2197	2614	3068

Table 5.5: Horizontal curve radius [m] at four different values of track cant (Assume Cant Deficiency, $\mathbf{h_d}$ = 100 mm)

Cant, h_t (mm)	Speed, V (km/hr)						
	140	160	180	200	220	240	260
100	1156	1510	1912	2360	2856	3398	3988
130	1006	1313	1662	2052	2483	2955	3468
150	925	1208	1529	1888	2284	2719	3191
180	826	1079	1365	1686	2040	2427	2849

Transition curve

For this research a linear variation of curvature (clothoids) and uniform variation of cant through the transition length is assumed; and the transition curvature shall coincide with superelevation ramps in both shape and position. 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:

$$\frac{1}{n} = \frac{\Delta h_t}{1000 * L_t} \leq \frac{1}{400} \qquad (2-26)$$

The length of the transition curve L_t, the larger value can be selected from the following formula using CEN provisional standard (from equation 2-36 and 2-37):

$$L_t \geq \frac{V_{max}}{3.6} * \Delta h_d \left(\frac{dh_d}{dt}\right)_{lim}^{-1} \qquad (2-36)$$

$$L_t \geq \Delta h_t \left(\frac{dh_t}{dx}\right)_{lim}^{-1} \qquad (2-37)$$

In the following table (Table 5.6) possible examples of transition curve length [m] for line of speed 180 km/h with both variation of cant and variation of cant deficiency of 100 mm which is summarized from Appendix D of Table D-2 and Table D-3.

Table 5.6: Transition curve length [m] for line of speed 180 km/h

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

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

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 $(dh_t/dx)^{-1}$= 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.

Gradient

Since large gradient results increased locomotive power, less freight train weight, reduced speed and line capacity, requirement of higher braking capacity, and/or larger signaling distances; restrictions for the amount of gradient are needed.

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.

Since the topography of Ethiopia is full of rises and falls, as the Ethiopia Road Authority feasibility study of some roads indicates, achieving minimum gradient is hard and leads to use up to 3.5% for mixed traffic line with speed up to 200 km/h, 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.

- Reducing line speed, and weight for locomotives of freight train; and
- Higher braking performance of vehicles and larger signaling distances.

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 and are also shown in Table 5-7 below:

$$R_v = \frac{V_{max}^2}{12.96 * a_v} \geq R_{v,lim} \qquad (2-42)$$

Where: $a_v = 0.22$ m/s² and 0.31 m/s² (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. Vertical alignment of stabling and service tracks shall not less than curves of radii 600 m on a crest or 900 m in a hollow. These are the limited minimum value, $R_{v,lim}$ of vertical curve radius in serviceable condition.

Table 5.7: Vertical curve radius [m] for different speeds of mixed lines with passenger trains

Speed, V (km/hr)	Vertical Radius, R_v [m]	
	Recommended Value ($a_v = 0.22$)	Minimum Value ($a_v = 0.31$)
80	2245	1593
100	3507	2489
120	5051	3584
140	6874	4879
160	8979	6372
180	11364	8065
200	14029	9956
220	16975	12047

Note:

Once basic elements of horizontal curve and vertical curve elements are determined; other elements of the curves to set out the curve are indicated in Appendix D, Part-1 of Figure D-1 and Figure D-2 respectively.

5.1.1. Comparison of the proposed geometric parameters with current design practice

After completion of this proposal work, a comparison was done to compare with current design practice on different railway track design projects in Ethiopia. For this purpose the Mekele – Haragebeya – Semera – Elidar – Tadjourah port was selected since the worst topography is found on this line on Mekele – Haragebeya section. This thesis was made according to the possible allowable provision of different railway standards and projects.

A typical preliminary design made by a consortium of OIA Infrastructure Developers – convergent business solutions (FZE) (2012) on Mekelle – Hara Gebeya – Semera – Elidar – Tadjourah Port railway line shows that significant sections have to be designed with a gradient greater than 1.5% since the topographical condition of the route is difficult. Especially on Mekelle – Hara Gebeya section, which is topographically worst section, a 25 km continuous gradient of 2.43% are allowed starting from station 171+1600 on Mekelle – Haragebeya section. But according to this thesis recommendation, the maximum length of continuous 2.0 mm/m (2.0%) gradient should not exceed 3 km. So the Ethiopia Railway Corporation (ERC) provision of 2.43% continuous gradient throughout the 25 km length is not advisable and needs special treatment on this section.

In addition on the design of horizontal alignment, a minimum value of 1000 m is allowed and this provision satisfies the minimum curve radius recommendation. And on Semera – Elidar – Tadjourah Port section at station 421+750, for a horizontal curve radius of 1000m; a track cant of 200 mm and a cant deficiency of 102 mm is suggested for the design of horizontal curve. But for mixed traffic line, according to this proposed guideline, a track cant value of greater than 160 mm shall not be recommended to avoid load displacement on floor of the wagon and high cant excess for slow moving freight train. As a result it is better to increase cant deficiency and reducing track cant. Possibly this can be rearranged to a track cant of 160 mm and cant deficiency of 142 mm by satisfying the required equilibrium cant.

5.2. Railway Track Structures

The type of track structure to be used in Ethiopia for rural line should be ballasted track and for urban should be covered (embedded) track. Of course the ballasted track have more advantages than covered track on that it is more economical, gives stiff support for the track and reduces noise due to its damping behavior. The slab track may be used if the economic consideration (initial investment cost) does not matter on the selection of the track type; but it is maintenance free track system. Whereas covered track is widely practiced in cities (LRT system) where space is limited, and road and pedestrian traffic may also need to use the area where the track is installed. In addition grooved rail profile type should be recommended when the track type is covered (embedded) track.

5.2.1. Rail

The rails, a pair of longitudinal steel members, have a primary function to guide the train in the desired direction; and to transfer the traffic loading to the sleepers. To provide this function the rail should resist wear and rolling contact fatigue (RCF) for the designed axle load and annual tonnage. Track is designed to a maximum axle load, i.e., the maximum total weight felt by a given axle (in the order of 20 - 40 ton/axle) which is further related to the weight of rails, density of sleepers and fixtures, train speeds and amount of ballast.

Rail Profiles

A rail shaped in section somewhat like an I-beam is commonly preferred to provide flexural stiffness and strength for plain line track. 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. The most commonly used rail profile, which should be selected, is flat-bottom rail also called Vignole rail.

Dynamic Impact Factor

The loads of dynamic impact factor depends basically sensitivity of the line to axle load and annual tonnage. Therefore high value of these parameters leads to use high value of dynamic impact factor. In addition rising axle loads and speed in the future also initiates to use high value. Even if the AREMA dynamic impact factor is safer than the other, it may need high weigh rail and as a result it can be expensive. But for future growth of axle load and annual tonnage, the rail may serve within the expected service life without defect. Therefore it is recommended to use AREMA dynamic impact factor, from Table 2-10.

$$\emptyset = 1 + \frac{5.21V}{D} \quad (5-3)$$

Allowable Bending Stresses

In Ethiopia, in addition to the topography, temperature variation is high in different parts of the country. So design of structures that highly affected by temperature variation, such as steel structures, should be designed considering thermal induced stress. So the allowable rail bending stress should consider thermal stress of the rail and can be expressed as

$$\sigma_{all} = \frac{(\sigma_y - \sigma_t)}{f_s} \quad (5-4)$$

$$f_s = f_{lat} \times f_{tr} \times f_{cw} \times f_h \quad (5-5)$$

Where:

σ_{all} − Allowable bending stress of rail

σ_y − Yield stress of the rail steel

σ_t − Temperature induced stress in the rail (longitudinal stress)

f_s − Combination of factor of safety for lateral bending of the rail (f_{lat}), track condition (f_{tr}), rail wear and corrosion (f_{cw}), and unbalanced superelevation of track (f_h).

At sever condition, the AREMA value of $f_{lat} = 1.25$, $f_{tr} = 1.25$, $f_{cw} = 1.15$ and $f_h = 1.15$ can be recommended; but these values are greater than 1 at any condition.

Steel Rail Grade Selection

The basic problem in rail service life is rail head wear and rolling contact fatigue (RCF). This problem can be solved by head hardening which can be achieved by increasing tensile strength of the rail since Brinell Hardness Number (BHN) of rail is directly related to tensile strength of the rail. For example, high cant deficiency contributes lateral wear of high rail and cant excess causes damage of low rail, for high axle load, so it should be better of selecting higher quality rail grades. But due to high investment cost of higher quality steel grade than standard steel rail strength, it is better of selecting rail that can carry the expected axle load and selecting appropriate maintenance method.

It is possible to recommend uniform rail grades, commonly standard rail grade i.e. R260 (R900 of UIC), for the whole range of curvatures but the wear of rail for low radius curves (up to 500 m or 700 m) may be high and as a result it needs high maintenance cost through the service life of the rail. The use of a rail grade of less than R260 should be avoided since the rapid growth of axle load and annual tonnage leads wear of rail before the service life.

Even if the use of high quality rail grades (harder rails), which are more resistant to wear and rolling contact fatigue (RCF) than standard grade rails, cannot be replaced by effective maintenance; in order to achieve a cost effective service life for rails, choice of rail steel grade and maintenance should be based on both technical and economic criteria. Technically the allowable bending stress of the rail, which can resist wear and RCF, should be greater than the bending stress due to wheel load; and economically the initial investment cost and the maintenance cost should be optimized (minimized).

The most appropriate steel rail grade, which can carry the axle load and is not much affected by wear and RCF particularly at low horizontal curve radius, should yield the lowest average annual overall cost. The average annual overall cost may be defined by the general formula, according to equation 2-47:

$$\mathrm{P} = \frac{\mathrm{C}}{\mathrm{n}} + \mathrm{M} \qquad (2-47)$$

Where: P – average annual overall cost, C – investment cost, n – service life of rails (in years), and M – annual maintenance cost.

Rail Section Design

Once the rail grade is selected, the rail section can be chosen using the design criteria of bending stresses of $\sigma_{\mathrm{max}} \leq \sigma_{\mathrm{all}}$; and it follows that $\mathrm{M_{max}}/\mathrm{Z_b} \leq \sigma_{\mathrm{all}}$. Thus, the required section modulus is

$$\mathrm{Z_b} \geq \frac{\mathrm{M_{max}}}{\sigma_{\mathrm{all}}} \qquad (2-48)$$

Once the section modulus of the rail section is determined, it is possible to determine the rail section from different railway standards, for example, from AREMA rail section.

When high lateral and/or extremely eccentric loads are encountered specially at low curve radius, it should be included in computation of rail bending moment.

In addition the maximum wheel load and/or wheel radius can be estimated using the shear stresses criteria of $\tau_{\mathrm{max}} = 0.3 * \mathrm{q_{mean}} \leq 0.3\mathrm{f_{yk}} = \tau_{all}$.

$$\mathrm{P} \leq \frac{\mathrm{f_{yk}^2} * 64\ \mathrm{Rb}\ (1 - \upsilon^2)}{\mathrm{E}} \qquad (2-49)$$

5.2.2. Fastening System

Fastening system which attach the rail to the sleeper have a function to properly attenuate and/or transfer loads and to retain the rail against the sleeper and resist the vertical, longitudinal, lateral and overturning movements of the rail.

Selection of fastening type

The type of fastening system that shall be selected in Ethiopia may be based on minimum cost with a good ability of fastening force, capacity of bearing ability and easily adjustable.

In addition, the selection system in Ethiopia shall be a matter to permit safe and smooth train operation. So the fastening system shall not be detachable easily since it may be stolen with the help of simple hand tools. As a result, if it is detached, the incapability of fastening system to resist lateral movement of rail especially at horizontal curves leads the rail to deflect from the normal alignment and train accident will occur.

The leaf spring fastening type may be the preferable one for railway system of Ethiopia since it cannot be detached easily and its good ability of fastening force.

Performance Test on Fastening System

The performance test on fastening shall be conducted to resist the effect of acting forces (i.e. vertical, lateral, rotational (both planes), and longitudinal force) to retain the rail against the sleeper and resist the vertical, longitudinal, lateral and overturning movements of the rail.

The tests that shall be done according to AREMA and European standards are repeated load test, longitudinal load restraint, determination of clamping force, stiffness of rail pad test, electrical resistance and torsional resistance. These tests shall be performed according to AREMA or European standards (EN 13481).

5.2.3. Sleeper (Crosstie)

The choices of concrete sleepers, more widely used in the world, are because of different reasons; basically it is according to economical advantages. In addition, in Ethiopia steel is relatively highly costy; 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 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 from Table 2.16. Therefore, the average pressure (P_a) at the tie bottom can be formulated as:

$$P_a = \frac{q_r}{b \times (l - g)} \leq P_{all} \qquad (5-6)$$

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.

Allowable average pressure on ballast (P_{all}) shall be determined with extensive experiment on different type of ballast material.

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 prestressed concrete (PSC) sleepers

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

For example according to Appendix D, Table D-5, considering other costs in addition to production cost, for instance defect due to transportation and installation, and transportation cost those increased with number of sleepers; the optimized sleeper that analysed can be selected with the dimension of 2.7 m length, 253 mm average breadth, 215 mm depth (height) and sleeper spacing of 580 mm. Even if the total cost per kilometer track length decrease as the sleeper length increase, according to analysis result, the positive bending moment at rail seat increase and then moment on fastening also increases. As a result the best quality of fastener that can resist the bending effect should be needed and may be costly than expected in the optimization. To avoid such condition a shorter length than 2.7 m can be selected and possibly 2.6 m can be with minimum variation of cost. Therefore, the dimension of sleeper with 640 mm spacing can be 2.6 m length, 295 mm average breadth (take 300 mm at rail seat) and the exact height of the sleeper at rail seat and at center of sleeper can be determined after design bending moment is computed.

Design Moment of Sleeper

The calculation of design moment should depend on actual rail seat load on the estimated spacing of sleeper, base width of sleeper and length of sleeper. The depth of the sleeper can be provided with different iteration until the design bending stress satisfies the allowable bending stress. So the design moment calculation shall be according to Australia standard.

Table 5.8: Design moment calculation at critical section of prestressed concrete (PSC) sleepers

Rail Seat Moment		Center Moment	
M_r^+ (kN.m)	M_r^-(kN.m)	M_c^+(kN.m)	M_c^-(kN.m)
$q_r\left(\frac{l-g}{8}\right)$	Max{$0.67M_r^+$,14}	$0.05q_r$ (l-g)	$q_r\left(\frac{2g-l}{4}\right)$

After the bending moment is calculated, the combined stress (σ_b) due to prestressing tendon and rail seat load can be computed using the cross section moment of inertia and prestressing tendon eccentricity of PSC sleeper (Lutch, 2009) and the concrete material property shall satisfy to resist these stress of PSC sleeper.

Design Procedure of PSC

Design of prestressed concrete (PSC) sleepers shall involve the following steps.

A. Assessment of loading at rail seat and pressure distribution (to determine effective bearing area)

B. Selection of base plan

C. Determination of sleeper spacing and sleeper width and length (by optimization technique)

D. Calculation of imposed bending moment at critical sections of PSC sleeper

E. Selection of suitable profile for the PSC sleeper and assigning depth of the sleeper

F. Providing number of prestressing tendon and computing eccentricity

G. Check permissible stresses in prestressing steel

H. Calculation of stress due to prestressing tendon and design rail seat load; and allowable concrete stresses at critical sections of PSC sleeper

I. Check whether the combined stress (σ_b) less than the permissible concrete stress (f_c) or not. If $\sigma_b \leq f_c$, the moment capacities of PSC sleeper for the design governing conditions of positive bending at the rail seat section and negative bending at the center section shall be checked. In addition positive bending at the center section and negative bending at the rail seat section shall be checked.

Note: The procedure of step F to I are verified in Appendix-D, Part-2, Section D.2.2, based on American Concrete Institute (ACI 318, 2008).

5.2.4. Track Ballast

Ballast Material

According to quarry sites investigation of ERA, Ethio-Djibouti railroad upgrade project and Ethiopia Railway Corporation (ERC), the type of possible materials selected for track ballast crushed aggregates are basalt (may include trachyte basalt and aphantic basalt), lime stone, and rheolite.

Tests for particle characteristics

Ballast material quality is defined by its particle characteristics and many tests have been done to define these characteristics. The optimum selected ballast index characteristics test can be categorized in to durability tests, shape tests, stability test, environmental impact and gradation.

Durability test

Los Angeles abrasion (LAA) test has long been used as the primary abrasion test; it is believed to measure a material's toughness or tendency for coarse breakage (fragmentation). But some engineers believe that it is not sufficient and Micro-Deval attrition (MDA) becomes another primary abrasion test carried out to measure the resistance of ballast to wear. These two tests can exactly define degradation of ballast and the impact of sudden shock loading can be considered by a test of aggregate impact value (AIV).

Therefore the recommended durability tests that shall be performed are LAA, MDA and AIV. The limited maximum value for this test shall be recommended according to the practice of different standards and it may on consideration of traffic (annual tonnage), environmental condition and cost of material delivered to the site.

Shape Test

The most common shape tests used not only in railway industry but also in highway projects are flakiness index (FI) and elongation index (EI) including Ethiopia. Therefore, flakiness index (FI) and elongation index (EI) shall be recommended to be performed in Ethiopia railway industry.

Stability Test

Specific gravity is the commonly used parameter of stability not only in Ethiopia (i.e. ERA and Ethio-Djibouti railway) but also in different railways which influences ballast unit weight. The determination of the amount of water absorbed by the particle is the part of this test. Water absorption is an indication of the rock porosity which relates to its strength as well as its tendency to break under freezing condition.

Environmental Test

The Sulphate soundness test shall be performed to consider the impact of weather on the ballast material. It is the practiced test in ERA for base course granular material.

According to the practice and recommendation of Consta JV (2007) for Ethio-Djibouti railway (CDE), the limiting value of ballast material test is tabulated below.

Table 5.9: Ballast material requirement characteristic tests, test standards and possible limited values

Ballast material test		Test standards (*)	Limiting value
Durability Test	Los Angele Abrasion (LAA)	ASTM C535 ASTM C131	< 22%
	Micro Deval Attrition (MDA)	BS EN 1097-1	< 7%
	Aggregate Impact Value (AIV)	BS 812-110	< 22%
Shape Test	Flakiness Index (FI)	BS 812-105-1	< 25%
	Elongation Index (EI)	BS 812-105-2	< 25%
Environmental Test	Sodium Sulphate Soundness	ASTM C88	< 5%
Stability Test	Specific gravity	ASTM C127	≥ 2.6
	water absorption	ASTM C127	< 1%

Note: (*) Familiar adoptions of test methods practiced in Ethiopia are commonly from ASTM or British Standard manuals since they are internationally accepted standards.

Ballast Gradation

The gradation of ballast plays a significant role in the strength, deformation, degradation, stability, safety and drainage of tracks. Therefore, an attempt shall be done to find a suitable range of particle size distribution which fulfills the objective of good strength (well graded) without significant reduction in permeability of ballast (uniform graded).

Consideration in ballast size limitation

- If ballast particles are larger than the maximum size (upper boundary of gradation), the particle may only be two or three stones between the underside of the sleeper and above the sub-grade which will be insufficient to properly distribute the load.
- Too many small crushed stones will however clog the ballast and reduce, in the longer term, its drainage properties.
- An excessive proportion of fines could impair, impede or even totally rule out certain desirable ballast properties.

Therefore, according to experience of Ethio-Djibouti and other railways, the particle of dimension exceeding 75 mm shall be limited to 25% by mass with regard to elongation criteria and similarly particle of dimension exceeding 100 mm shall be limited to 2%. The amount of fine particles passing on sieve size No-200 (0.075 mm) shall be limited to 1% according to experience of Ethio-Djibouti and other railways. In addition a crushed aggregate particle of size less than 22.4 mm that contribute to drainage problem shall be minimized as possible.

Table 5.10: Gradation of ballast material [moderately graded ballast]

Square Sieve Size (mm)	75	63	50	37.5	31.5	22.4	19	12.5	0.075
Percent passing (by mass)	100	90-100	60-85	20-55	15-25	5-15	0-7	0-3	0-0

Note: The tabular value is shown as an envelope in Figure 5.1.

A moderately graded ballast material, which the lower boundary limit is approached to gradation of AREMA, have a coefficient of uniformity, $C_u = D_{60}/D_{10} = 50/22.4 = 2.23$ which is between 2.2 and 2.6. This gradation can provide sufficient permeability and ballast material breakage decrease with respect to uniform graded ballast, but at a long term the ballast may suffer from drainage problem. To satisfy this property the ballast, the following criteria shall be included.

- Ballast material shall be free from fines (fouling) otherwise the fines content shall be minimized.

– An appropriate drainage system shall be constructed along the track.

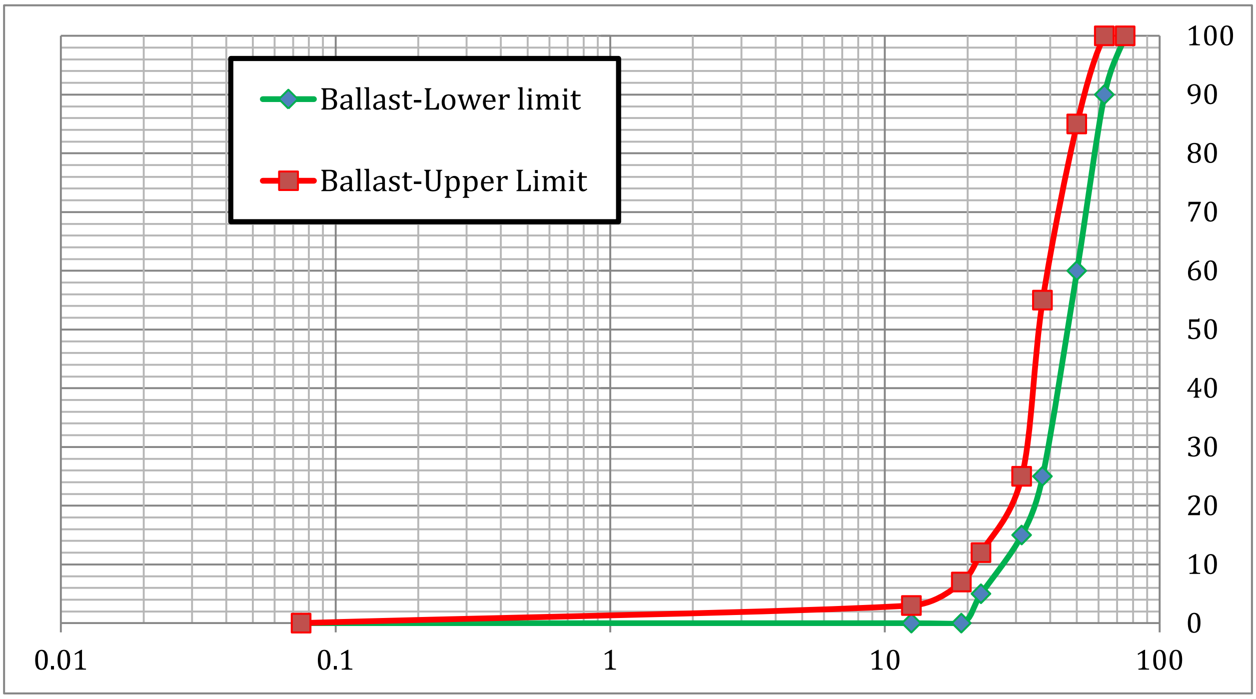


Figure 5.1: Possible gradation of ballast material [moderately graded ballast]

Uniformly graded ballast, which has a coefficient of uniformity less than 2, can be recommended for high permeability requirements and in addition this may have void spaces for storage of potential fouling. But the ballast may suffer from degradation, low shear strength as a result track instability, track settlement and etc. Therefore, the cyclic triaxial tests must be done to determine exactly the strength as well as degradation and settlement characteristics of the recommended ballast gradation.

Ballast Depth

The ballast section depth including subballast shall be determined according to AREMA equations that have been developed, Talbot equation.

$$h = \left[\frac{16.8 * P_m}{P_c}\right]^{0.8} \qquad (2-56)$$

Where: h= granular layer thickness (ballast + subballast) (in), P_c = Allowable subgrade pressure (psi), and P_m = Vertical stress applied on the ballast surface (psi).

The required depth shall depend upon the maximum speed of trains, the maximum axle loads carried and the gross annual tonnage expected; but it shall be greater than the recommended limit ballast depth. The recommended minimum ballast section depth for concrete sleeper track shall be 300 mm from experience of Railway Group Standards (RGS).

5.2.5. Track Subballast

In most cases, standard gradations for dense graded aggregate and aggregate base course may meet the requirements for use as sub-ballast. Applying the filter principle used in drainage to the grading of the sub-grade material will determine the grain size distribution of the sub-ballast.

Subballast Material Selection

The availability and economic considerations are the main factors considered in the selection of rock quarries not only for ballast but also for sub-ballast materials. Materials to be used as subballast can be granular materials, boulders, crushed weathered rocks and/or fresh and crushed rocks which are available in Ethiopia. In addition the materials identified for ballast can also be used as good sub-ballast material.

Subballast material Specification

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

Tests of Subballast Material Properties

Even if the durability requirements are not as severe as for ballast because the subballast particles are smaller and the stresses are lower, LAA test is performed to consider the degradation characteristics of the aggregate.

CBR tests (a comparative not a characteristics measure) especially for granular materials shall be performed to compare the bearing capacity of the material with that of well graded crushed stone (which should have CBR of 100%).

The Atterberg Limits are performed only on that portion of a soil (cohesive) that passes the 425-μm (No. 40) sieve. Therefore, the relative contribution of this portion of the soil to the properties of the sample as a whole must be considered when using these tests to evaluate properties of a soil which can affect strength and settlement characteristics.

The mechanical compaction (moisture-density relation) shall be performed since it is one of the most common and cost effective means of stabilizing soils.

Specific gravity is a common test that shall be performed for the purpose of stability of the track. The Sulphate soundness test shall be also performed to consider the impact of weather on the ballast material.

Depending on the practice of ERA and Ethio-Djibouti Railway (CDE) line upgrade project that are also practiced in different railway systems; the material test, test standard and limiting value of each test is tabulated below.

Table 5.11: Sub-ballast properties and test methods

Material test	Test Standard	Comment
Moisture Density Relation ($\geq$ 70% particle pass on 19 mm sieve)	ASTM D 1557	MDD and OMC
Atterberg Limits (Particle passes on sieve 0.425 mm)	ASTM D 4318	LL and PL $PI = LL - PL \leq 6$
LAA	ASTM C 131	LAA $\leq$ 30%
CBR (for particle size $\leq$ 19 mm)	ASTM D 1883	CBR $\geq$ 30%
Sodium Sulphate Soundness	ASTM C 88	Loss $\leq$ 12%
Specific Gravity (γ)	ASTM C 127	$\gamma \geq 2.40$

Where: MDD- Maximum Dry Density, OMC-Optimum Moisture Content, LL-Liquid Limit, PL-Plastic Limit, PI-Plastic Index, LAA-Los Angeles Abrasion and CBR-California Bearing Ratio.

Possible recommended gradation of subballast

Even if sufficient fines (amount of material passing the 0.425 mm sieve) are needed to produce a dense material when compacted, the fines content shall be limited for drainage purpose. Depending on the permeability requirements for drainage, the preferable recommended limit of fine particles ($\leq$ 0.075 mm) shall be between 0 to 5 percent.

The gradation of subballast aggregate shall be performed according to AREMA way of determination since it is reasonable for filter principle of drainage criteria. So the gradation of sub-ballast material can be determined by use of the grain size distribution of the sub-grade.

Possible subballast gradation value computation

a. Assumption
   - The subballast contains a well graded angular particle; therefore, for filter criteria ratios R_{50} is 9 to 30 and R_{15} is 6 to 18.
   - Brown silty clay soil with gravel subgrade gradation as tabulated below

Table 5.12: Assumed optimum gradation of brown silty clay soil with gravel subgrade material for railway track

Sieve size (mm)	9.5	6.7	4.75	2.36	1.18	0.425	0.075	0.045
% passing	100	99	96.5	88	75	50	15	10

b. Subballast material size determination for pass of 15 and 50 percent

Table 5.13: Subballast size determination satisfying filter principle drainage criteria

Passing point (%)	Size of subgrade particle (mm)	Subballast ratio		Subballast Size (mm)	
		Min.	Max.	Min.	Max.
15	0.075	6	18	0.45	1.35
50	0.425	9	30	3.825	12.75

c. Possible subballast gradation that satisfy filter criteria can be computed by setting the upper and lower boundary sizes by iteration

Table 5.14: Subballast gradation that satisfy filter criteria for nominal size of 37.5 mm

Sieve size (mm)	50	37.5	19	12.5	6.7	4.75	1.18	0.425	0.075
% passing	100	95-100	70-95	45-80	35-60	30-55	10-25	5-14	0-5

Note: The tabular value is shown as an envelope in Figure 5.2.

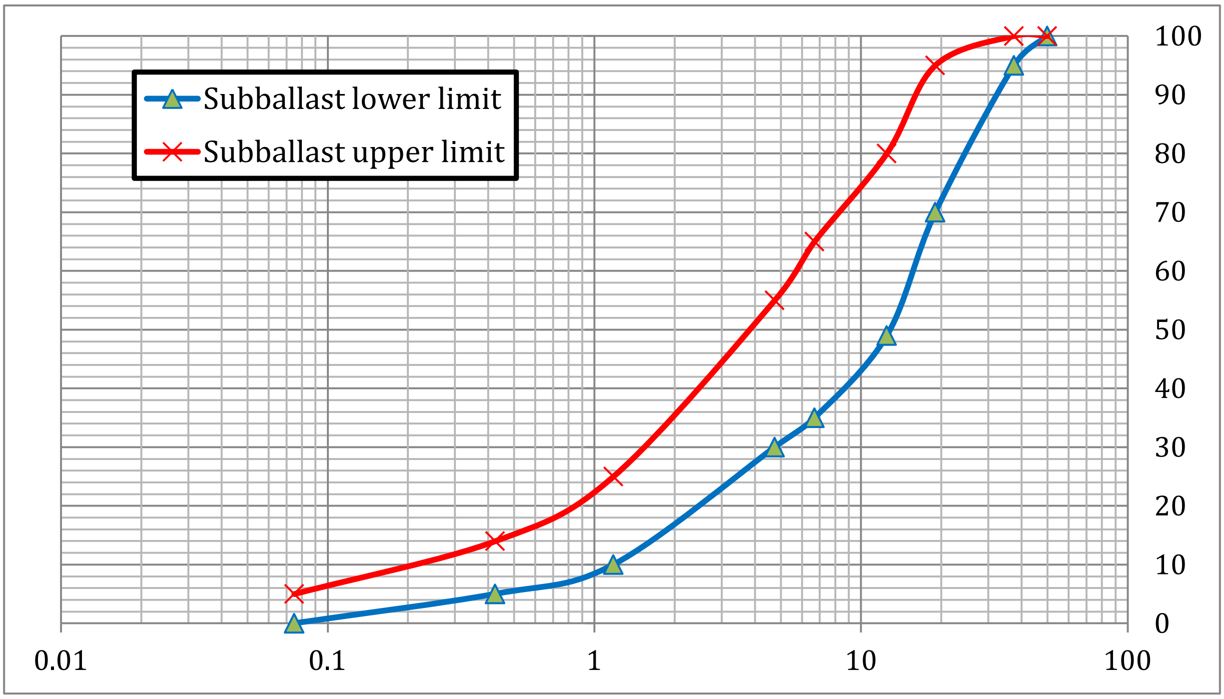


Figure 5.2: Gradation of subballast satisfying filter principle drainage criteria

Subballast Section Depth

The subballast section depth ($\mathbf{h_{sb}}$) shall be determined according to AREMA equations that have been developed, Talbot equation. Since the distribution of loads to depth is approximately the same regardless of the granular material, the combined depth of sub-ballast and ballast is calculated as a single unit to develop the pressure on the subgrade. So the required depth shall be depending upon the maximum speed of trains, the maximum axle loads carried and the gross annual tonnage expected; but it shall be greater than the recommended limit subballast depth.

The recommended minimum subballast section depth shall be taken as 150 mm from experience of AREMA and recommendation of researchers to reduce the depth of ballast material as a result this intend reducing cost of the track.

The appropriate side slope shall be provided to facilitate drainage from the top of the roadbed construction.

5.2.6. Subgrade Soil

Subgrade is one of the most crucial parts of embankment fills or natural surface just below the subballast of railway track structure.

I. Subgrade Soil Types

The subgrade is often the weakest substructure layer and the types of material which are acceptable for use as a railway subgrade and their different characteristics are:

a) Material in place - whenever the track way will be in a cut section, subgrade will be in original ground. Soil in this condition may not have the strength to support the above track structure and in order to achieve desired strength, these soils must be compacted.
b) Imported material - subgrade material consisting of imported material is called “borrow material”. This material can come from regular excavation from another area in the project, from commercial sources, or from local pits or quarries. In addition it consists of removing problematic soils and replacement of good quality soils to provide stable foundation material.
c) Treated material in place - for some soils, simply compacting will not produce the desired strength needed to support structures. In these cases it can be very cost effective to stabilize the subgrade with lime, cement, fly ash or a combination of them to provide a solid foundation for the upper structure.

II. Subgrade Soil Characterization Tests

The following soil index property laboratory tests shall be undertaken on subgrade soil samples;

- Particle size distribution (Sieve analysis and Hydrometer test) - are needed to check the material grain size distribution whether satisfying the coefficient of uniformity greater than 5, i.e. $C_u = \frac{d_{60}}{d_{10}} \geq 5$ or not. In addition it is used to identify the type of stabilizer to be selected if it is problematic soil.
- Specific gravity- is performed to identify the capability of the subgrade soil to make the track stable.
- Atterberg limit tests- are performed only on that portion of a soil (cohesive) that passes the 425-μm (No. 40) sieve and these tests are used to evaluate properties of a soil which can affect strength and settlement characteristics of the subgrade. A high Atterberg plastic limit (PL) value and a minimum plasticity (Plastic Index-PI) are recommended for railway subgrade soil.
- Soaked CBR test- CBR test is used as an index of soil strength and bearing capacity. According to experience of ERA subgrade shall not be constructed on soils of CBR less than 3% (≤ 2% for arid area).
- Moisture-density relationship (compaction) test- is one of the most common and cost effective means of stabilizing soils and can be performed with or without additives; and this test is performed with objective to check the strength of subgrade soil to support upper track structures. The main advantage of compaction is increasing density; and in turn the strength of the subgrade layer also increases.

Table 5.15: Subgrade soil Properties and test methods

Material test	Test Standard	Comment
Moisture-Density Relationship	ASTM D 1557	MDD and OMC
Atterberg Limits (Particle passes on sieve 0.425 mm)	ASTM D 4318	LL and PL PI = ≤ 12
CBR (for particle size ≤ 19 mm)	ASTM D 1883	CBR ≥ 3%
Material Finer than No. 200 Sieve	ASTM C 117	Hydrometer test
Specific Gravity (γ)	ASTM C 127	γ, depends on soil type

III. Subgrade Layer Improvement

The problematic subgrade soil shall be replaced or improved to achieve the required strength characteristics of subgrade layers.

Procedure in the improvement of problematic subgrade soil:

- Material sampling from the investigated soil on the track line
- Evaluating the properties of given sample soil (different tests)

- If the soil does not satisfy the engineering property to be used as subgrade, select low cost locally available materials to replace the in place problematic soil.
- If the locally available material also cannot satisfy subgrade property, ignore it and consider the in place soil for stabilization.
- Deciding the lacking property of the in place soil and select effective and economical method of soil stabilization
- Designing the stabilized soil mix for intended stability and durability values
- Check the performance of the improved/stabilized test

Subgrade Soil Compaction

A well compacted subgrade material acquires improved strength, stiffness and bearing capacity; more resistant to moisture penetration; and less susceptible to differential settlements. The strength of the subgrade support leads to provide the lesser the thickness of the upper structure of the track and as a result the more economical of the ballast and subballast layer. Therefore; to achieve this objective, it is recommended that specially the top 25cm of all subgrades shall be compacted to a relative density of at least 98% of the maximum dry density achieved by ASTM Test Method D 1557 may be specified.

According to recommendation of LVR manual of ERA (2011) part-D, impact compaction provides an alternative to conventional compaction plant for undertaking compaction at low moisture contents, since soils compacted at low moisture contents will have high air voids. Impact Compactors are non-circular, relatively high-energy 'rollers', typically three- sided, four- sided or five- sided.

Stabilization of subgrade soil

For some soils, simply compacting will not produce the desired strength needed to support upper track structures and as a result it can be very cost effective to stabilize the subgrade. The subgrade materials to be stabilized by different mechanisms (methods) from experience of Ethiopian Road Authority (ERA) and the newly developed manual for low volume roads (LVR) are:

- Expansive soils (dark or light grey clay)
- Collapsible sands
- Dispersive/erodible soils
- Saline and micaceous soils
- Low-strength soils

Selection of Stabilizer

The selection of stabilizer in Ethiopia basically shall depend on the type of problematic soil to be stabilized (basically grain size distribution and plastic characteristics), the cost of the stabilizer, and mechanism of stabilization.

The experience of Ethiopia Road authority showed that both mechanical methods and chemical additions are practiced to stabilize problematic soils.

The simplest and most commonly used method of mechanical stabilization, which shall be recommended, is accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification.

The use of geosynthetics (a class of geomaterials) including geotextiles, geogrids, geocomposites, and geomembranes which is a method of mechanical stabilization may be expensive and incapable to perform in Ethiopia for huge projects like railroad construction.

The chemical additives of considering the economic capacity of Ethiopia can be also applied. The experience from ERA and AACRA indicated that the cost effective stabilization types such as Any-Way's Natural Soil Stabilization (ANSS), CON-AID Chemical, Zyme-tech or other chemicals can be applicable for railway track subgrade in addition to lime and cement stabilization techniques.

IV. Subgrade Layer Design

Since the subgrade is often the weakest substructure layer, a combined ballast and subballast thickness is required to reduce the pressure on the subgrade to a level that produces an acceptably small deformation from the repeated train loading for the desired design life. The thickness of ballast and subballast layer depends on allowable subgrade bearing pressure that basically based on the soil condition. The vertical stresses that can be transferred to the subgrade layer must be lower than the allowable subgrade bearing pressure capacity.

This vertical pressure on the subgrade layer resulting from sleeper loading, the load transfer through the ballast layer should be investigated, and therefore Talbot's empirical equation (equation 2-52) can be used for the calculation of vertical pressure on the subgrade layer.

$$P_c = \frac{16.8 * P_m}{h^{1.25}} \qquad (2-52)$$

Where: h= granular layer thickness (ballast + subballast) (in), P_c = allowable subgrade pressure (psi), and P_m = vertical stress applied on the ballast surface (psi).

The upper surface of the subgrade shall have a crossfall sufficient to direct runoff towards the track drain. Usually a crossfall between 1 in 20 and 1 in 40 is necessary to meet this requirement.

Protection or Capping Layer

Protection or capping layer is provided to protect the subgrade layer from excessive overburden stress/load/ (when the formation level is found very weak, usually CBR < 3) and/or to avoid intrusion /mixing/ of coarse materials from the subballast into the subgrade. In addition its provision is recommended whenever it is found necessary particularly when the ground water table (GWT) has significant influence on the strength of the subgrade. To fulfill such functions, the size of the protection layer shall be finer than subballast layer and at least 20 cm thickness shall be recommended at any case above subgrade layer.

The gradation of protection layer shall satisfy the filter principle of drainage criteria. So the gradation of protection layer material can be determined by use of the grain size distribution of the sub-grade from lower limit boundary of subballast gradation.

According to Tables 5.10, 5.11 and 5.12; the possible gradation of protection layer that satisfy filter criteria is tabulated as below.

Table 5.16: Protection layer gradation that satisfy filter principle drainage criteria

Sieve size (mm)	37.5	19	12.5	6.7	4.75	2.36	1.18	0.425	0.075
% passing	100	70-85	45-75	30-65	25-55	20-35	10-20	5-10	0-5

5.3. Railway Safety System Consideration

5.3.1. Railway Safety Measures

Railway safety issues can be viewed in relation to railway infrastructure, mechanical condition (signal failures, equipment failures and train control), human effect and external effect (weather related and obstruction).

Therefore, careful planning, construction and operation schemes are essential to keep the railway system safe and reliable, considering the large number of passengers on board.

The safety issues related to infrastructures are basically quality control of railway infrastructure. Therefore, the quality control of the track infrastructures, which include geometry of the track and track structure in this study, are expressed according to proper design of the infrastructures to achieve the required capacity.

The quality of track geometry to achieve the expected deign speed of train and to maximize axle load, the track geometry shall

- Have minimum gradient and if possibly high horizontal curve radius
- Be carefully engineered around curves to 'bank' the outside rail and counter lateral forces to maintain an even weight distribution to both rails.

The main quality control of track structure is to transfer the design axle load to lower level of the formation layer and as a result appropriate quality of track structural materials shall be used to achieve smooth functioning of track structure.

According to discussion and recommendation of previous sections, the following points are summarized for railway safety related to track infrastructures.

- The rail shall be wear resistance to guide the train smoothly in the desired direction.
- The sleeper shall have optimum bottom area to transfer wheel load from rail to ballast without overstress of ballast; in addition the sleeper itself shall have desired strength to transfer this load without failure.
- The fastening system shall have good ability of fastening capacity and bearing capacity; but shall not be detached easily since it may be stolen with the help of simple hand tools.
- The substructure layers shall satisfy the strength and drainage characteristics, to avoid differential settlement of track structure. To achieve this objective, especially sufficient depth and good quality of ballast and subballast layer are needed. In

addition the subgrade layer shall be well compacted; and shall be well treated if the subgrade soil is problematic soil.

Mechanical problems such as signaling failure, equipment failure and train control failure are the other safety issues that can be control by controlling quality of devices and equipments. The train control quality is expressed in terms of breaking capacity (stopping distances) even if for trains to safely travel on a railway, trains must be provided with sufficient distance in which to stop. Whereas the signaling quality control, especially at highway-railway level crossing, is expressed in terms of providing uniform and constant function at every condition.

External factors related safety issues can be controlled by continuous monitoring of the railway track to investigate the problems to be occurred. In addition the track shall be free from obstructing objects on the track and continuous monitoring system shall be recommended in the railway track system.

5.3.2. Signaling and Communication

Communication and signaling systems are the main important systems used in safety system of railway operation. According to recommendation of China Railway Group Limited (2009) to Addis Ababa LRT project study report, the following points should be covered in communication and signaling system of the railway system of Ethiopia.

Communication System

The communication system of Ethiopia railway projects shall be designed for the purpose of train dispatching, power distribution, safety precaution, daily operation, and maintenance. The Communications System shall provide the necessary subsystems to support the total operational requirements. And this system should comprise of data transmission network, telephone, closed-circuit television monitoring system, wireless communication and related telecommunication ancillary facilities.

Data transmission network is a complex network platform; its main function is to provide with data channel for the Power Supervision Control and Data Acquisition （PSCADA）, IP telephone, and closed-circuit television monitoring system (CCTV), and to provide with switch in condition for the signal control data along the line.

Telephone system shall provide operational management, daily working, equipment maintenance, business connection and data exchange for the project, and it will be connected to municipal public telephone network.

The purpose of CCTV is to supervise the condition of each station, lines and municipal roads. On the one hand, to supervise the inflow and traffic condition of the station and crossing, on the other side, to provide evidence for traffic and security incidents happened at this area, to avoid various damages.

Wireless communication system is an important supplementary means for safety operation, improving transport efficiency and management level, and improving service quality. The system shall able to provide with wireless communication service for all kinds of coordinators, train driver and conductor, operator and staff along the line.

Signaling

According to different railway experiences concerning municipal lines and operational modes, the principle of train operation for the project is based on drivers experience and skill. Traffic evidence shall be displayed by means of signals at railroad switches, as for other places, security distance between trains shall be decided completely by the driver.

The signaling system may include track circuits signal system to suit ballasted track. The signal system, which includes the functions of signal protection, grade crossing protection, and traffic signal coordination, shall be designed to allow vehicles to meet the required line capacity, while providing a safe and operationally flexible system.

5.3.3. Railway Safety Management System

The railway safety measure is necessary to alleviate problems in railway system, reduce fatal accidents and the socio-economical impact.

Accident Assessment and Safety Program

The railway safety requires to set up an independent accident investigation body that shall notify the agency of the railway any investigations opened as well as to send the full investigation report when the investigation is closed.

The railway safety assessment method of safety management system (SMS) and engineering safety management (ESM), which are discussed previously and indicated in Appendix D Part-3, can be applied to enhance safe railway travel in Ethiopia.

5.4. Recommended Platform Dimensions

Station platform dimensions, a raised structure or area within a station, can be recommended for station platforms of both passenger and freight boarding. The Chinese code of standard recommendation is reasonable for both passenger and freight boarding system for different platform level and it is possible to recommend this standard for Ethiopia railway system [China Railway Group Limited, 2009]. It shall be located on track gradients not steeper than 1 in 500 wherever reasonably practicable. Where this is no reasonably practicable, appropriate risk control measures shall be agreed with the station operator.

Table 5.17: Recommended platform dimensions for standard gauge track for Ethiopia railway system

Boarding/ Alighting	Height of Platform edge from top of rail (mm)			Offset distance from adjacent track (mm)		
	Low level	Ordinary	High level	Low level	Ordinary	High level
Passenger	300	500	1250	1000	1000	1000
Freight	-	1100	≤4800	-	1000	1100

5.5. Switches and Turnouts

Switches can be operated either of the power operated switch and lock movements; electrically locked hand-operated machines; or hand-operated trail able switch stands, depending on the location and purpose of the switch. For LRT systems, power operated switch and lock movements are the preferable one since the frequency of the train is high. Whereas hand-operated trail able switch stands shall be provided for conventional railway lines in rural areas. Switch-protective signal should be installed at railroad switch area, and the driver shall operate the train as per instruction of protective signal when passing the railroad switch area. In addition crossing signals shall be located at level crossings, and it is only oriented to the forward direction of the train, the driver shall control the train as per instruction of the crossing signals.

At normal condition a turnout of at least number 1:12 shall be selected to allow smooth transition from normal track to the diverging track since as the turnout numbers become flat, the turnout length become long; but at difficult condition the sharper turnouts may be allowed up to 1:7. Depending on the speed of the train, simple turnout with movable nose frog (for speed greater than 160km/h) and movable external locking device (for speed greater than 120km/h) can be adopted.

6. Conclusion and Recommendation

6.1. Conclusion

This research attempted to set guidelines for track infrastructure subsystem of the railway system of Ethiopia basically concerning on track geometric parameters and track structures. The standard guidelines for different railway track elements can be done according to the country topographical condition, climatic condition, economic criteria, design load, and construction material availability.

1. The provision of cant dimensional value depends on the topographic condition of the alignment whereas cant deficiency and cant excess are an economic balance between the low rail damage caused by the heavy freights and the high side gage face wear caused by the passenger (fast) trains.
2. Through difficult topographical condition of Ethiopia a gradient up to 3.5% may be used with a significant limitation of continuous gradient length, speed and weight of freight train by allowing a high traction power, high braking performance of vehicles and larger signaling distances.
3. Due to high investment cost of higher quality steel grade; standard steel rail strength i.e. R260 (a minimum Brinell Hardness Number (BHN) of 260) is recommended for all lines including for the whole range of curvatures. But low radius curves are highly suffering from wear and as a result it needs high maintenance cost through the service life of the rail.
4. The spacing of sleeper and dimension of sleeper (base width and length) is optimized in consideration of satisfying the allowable bearing pressure and the optimization analysis needs estimation of cost of sleeper.
5. The types of test that can define the characteristics of ballast material in Ethiopia are durability test (Los Angele Abrasion, Micro-Deval Attrition, and Aggregate Impact Value), shape test (Elongation Index and Flakiness Index), stability test (specific gravity and water absorption), environmental impact test (sodium sulphate soundness) and gradation test.
6. A moderately graded ballast material with a coefficient of uniformity between 2.2 and 2.6, which can provide sufficient permeability and decrease ballast material breakage with respect to uniform graded ballast, is recommended for Ethiopia railway system.

7. Tests selected to be performed to define characteristics of subballast materials are moisture density relation ($\geq$ 70% particle pass on 19 mm sieve), Los Angele Abrasion, Atterberg limits (for particle passes on sieve 0.425 mm), CBR (for particle size $\leq$ 19 mm), sodium sulphate soundness and specific gravity.

8. The gradation of subballast material can be determined according to filter principle of drainage criteria using the grain size distribution of the sub-grade.

9. Tests selected to be performed to define characteristics of subgrade materials are moisture density relation, Atterberg limits (for particle passes on sieve 0.425 mm), CBR (for particle size $\leq$ 19 mm), material finer than No. 200 Sieve (hydrometer test) and specific gravity.

10. Weak subgrade soils are either replaced or improved to achieve the required strength characteristics of subgrade layers; and the methods to improve such weak soils are compaction and use of stabilizing agents. The selections of stabilizing agents in Ethiopia are based on the type of problematic soil to be stabilized and the cost of the stabilizer.

6.2. Recommendation for Future Works

The present work has attempted to standardize guidelines for track geometry parameters (cant, cant excess, cant deficiency, horizontal curve radius and vertical curve radius); track structures (rail, fastening system, sleepers, ballast, subballast and subgrade layer); and a little about corresponding safety system. However, due to financial constraints and time limitations the present research work did not cover detail information on railway system.

In view of this work, it would be desirable to consider the following recommendations for the future work for the development of modern railway system in Ethiopia.

A. Full scale experiments should be set up and carried out in Ethiopia to determine the optimum (sufficient) fine content for railway subballast to make the layer denser and satisfy filter criteria as well as to determine appropriate ballast gradation of sufficiently satisfying both permeability and strength.

B. Since many of the possible problems with the substructure are connected to water, the drainage system (usually considered as a part of the substructure) should need a special concern and detailed study.

C. The safety issues related to mechanical problems such as signaling failure, equipment failure and train control failure should be taken as a critical issue in future studies. And in addition the contribution of track structure failures to safety problem in railway system needs a detail investigation.

D. The contribution of speed of train, negligence or poor safety systems/regulation, overcrowding and congestion, and unauthorized access to accident should need a detail future work in safety system of railway system of Ethiopia.

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8. Appendixes

Appendix A: Definition

Horizontal Curve is a curve in the track used to join two alignments in plan

Vertical Curve is a curve in the longitudinal profile of track used to join two gradients together.

Transition Curve is a curve of varying curvature; it is normally provided between two circular curves, each of different radius, or between a circular curve and a straight.

Cant is the amount by which one running rail is raised above the other running rail; measured at the center of rail head

Equilibrium Cant is the amount that it is necessary to raise one running rail above the level of the other running rail to obtain equilibrium at a nominated speed

Cant Deficiency is the difference between actual cant and the theoretical cant that would have to be applied to maintain equilibrium at a nominated speed

Cant Excess is the difference between actual cant and equilibrium cant when the actual cant is higher than the equilibrium cant.

Cant Gradient is the amount by which cant is increased or decreased in a given length of track, e.g. 1 in 1200 means that a cant of 1mm is gained or lost in every 1200mm of track.

Quasi-static curving is curving at constant speed, radius and cant on perfect track geometry.

Tensile strength is the applied stress required to cause failure is greater than the yield stress and is generally defined as tensile strength.

Rolling Contact Fatigue (RCF) refers to range of defects that occur mainly due the development of excessive shear stress at or close to the wheel / rail contact which exceed the shear limit of rail material.

Rail Wear is the deviation of rail from the original rolled cross-sectional profile due to friction and abnormal heavy load

Dynamic Impact Factor is a corrective factor to compensate for dynamic as well as impact effects of wheel load resulted from wheel and rail surface irregularities.

Appendix-B: Abbreviations

ACI = American Concrete Institute

AREMA = American Railway Engineering and Maintenance-of-Way Association

ASTM = American Society for Testing of Materials

BR = British Railway

CBR = California Bearing Ratio

CDE = Chemin de fer Djibouto-Ethiopien (Ethio-Djibouti Railway)

CEN = Comité Européen de Normalisation (Committee for European Standardization)

CN = Canadian National Railway

CWR = Continuous Welded Rail

DB = Deutsche Bahn AG (German Railway)

ERA = Ethiopia Road Authority

ERC = Ethiopia Railway Corporation

FRA = Finnish Rail Administration

JR = Japan Railways

LL = Liquid Limit

LVR = Low Volume Roads of ERA

TSI = Technical Specification of Interoperability

MDA = Micro-Deval Attrition

MDD = Maximum Dry Density

NCHRP = National Cooperative Highway Research Program

OMC = Optimum Moisture Content

ORE = Office of Research Experiments (now UIC)

PI = Plasticity Index

PL = Plastic Limit

RGS = railway Group Standard

SNCF = Société Nationale des Chemins de Fer Français (France Railway)

TGV = Train Grande Vitesse (High Speed Train)

UIC = Union Internationale des Chemins de Fer (International Union of Railways)

UK = United Kingdom

US = United States

USCS = Unified Soil Classification System

Appendix C- Cited Maps, Figures and Tables

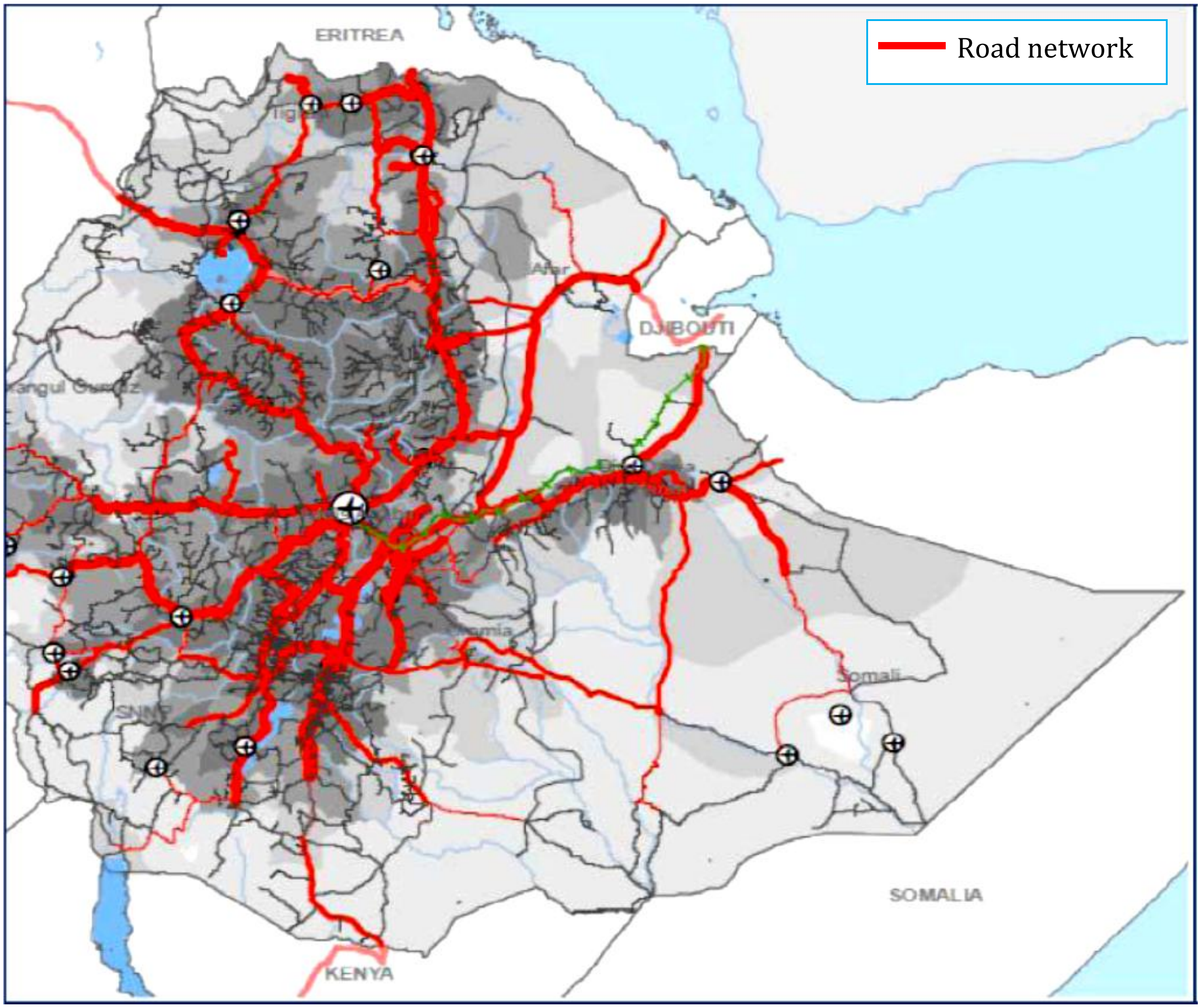


Figure C-1: Ethiopia national road network map [Ibrahim Worku, 2011]

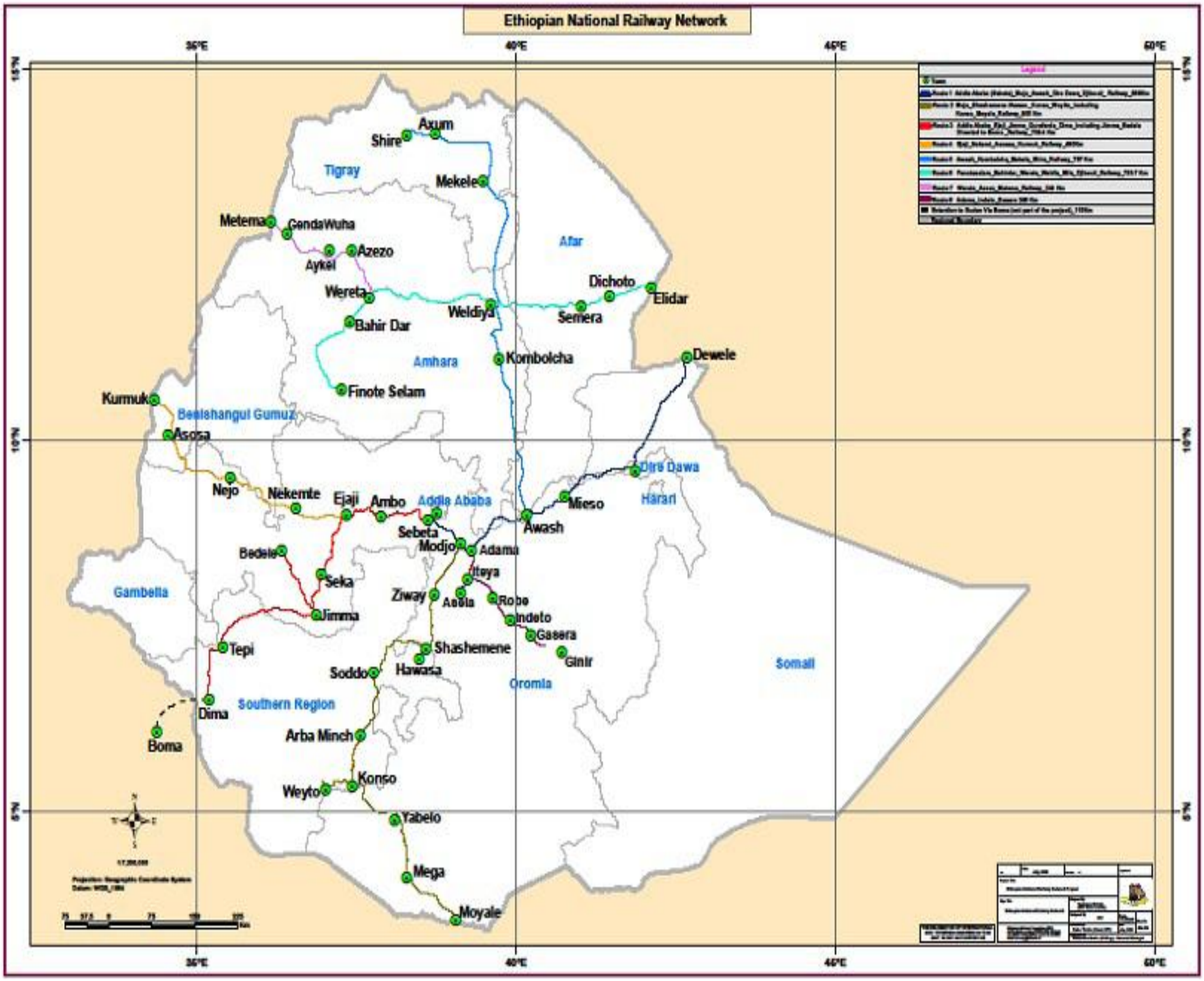
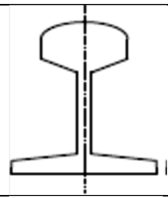
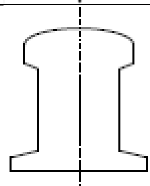
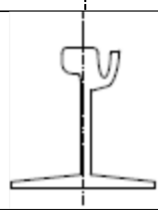
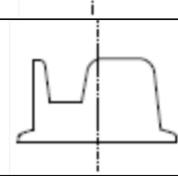
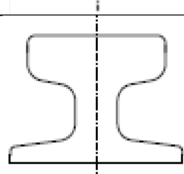


Figure C-2: Ethiopia national railway network map [ERC, 2011]

Table C-1: 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

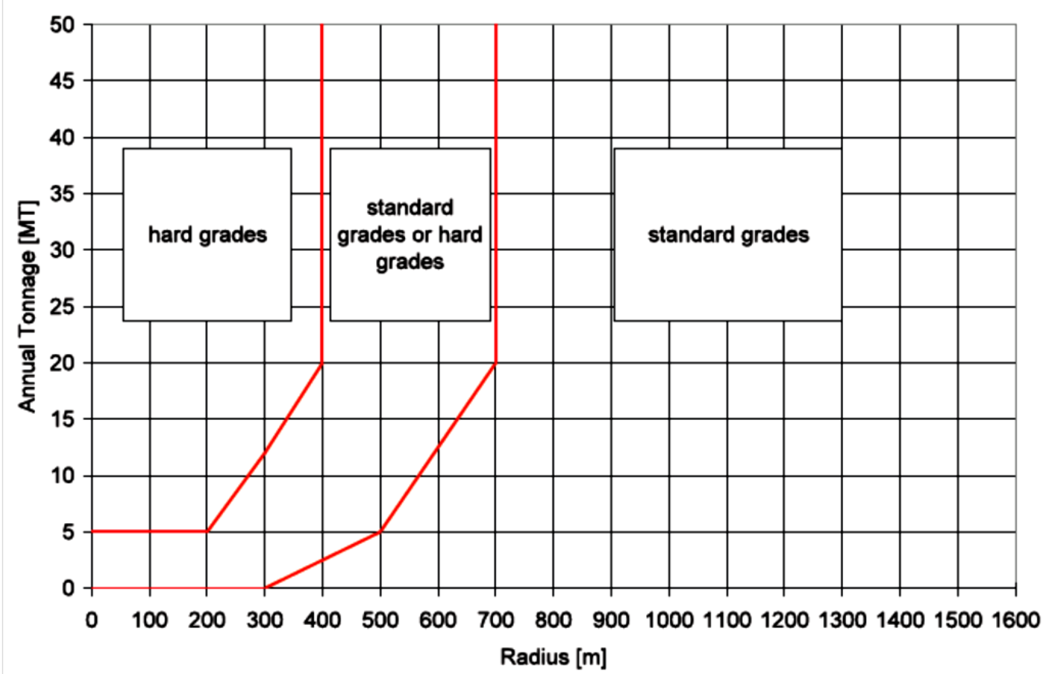
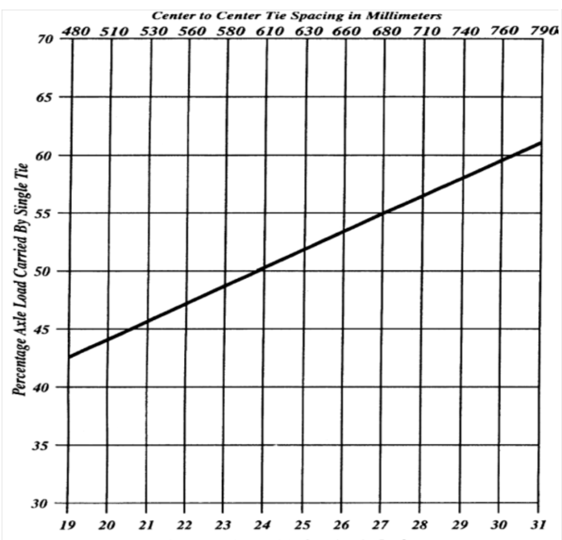


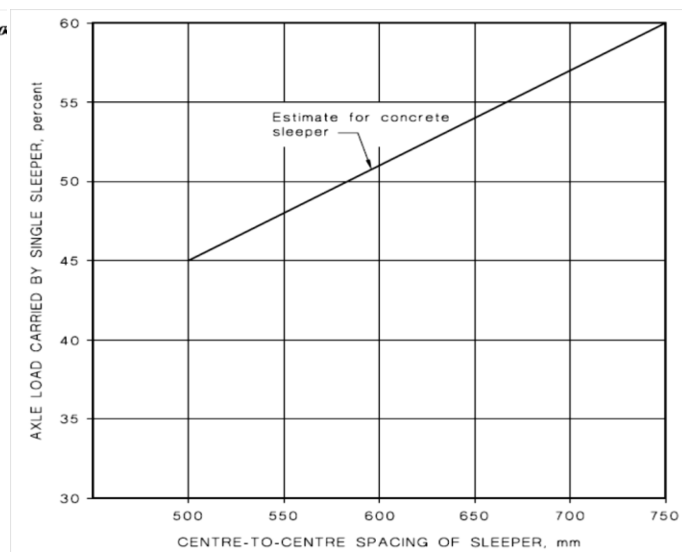
Figure C-3: Recommendation for the use of standard and hard steel grades according to UIC leaflet 721 [Pointner, 2010]

Table C-2: European current pearltic steel rail grade properties [Pointner, 2010]

Rail grade	R200	R220	R260	R260Mn	R320Cr	R350HT	R350 LHT	R370 LHT
Hardness (HB)	200-240	220-260	260-300	260-300	320-360	350-390	350-390	370-400
Min UTS (Mpa)	680	770	880	880	1080	1175	1175	
Description	C-Mn	C-Mn	C-Mn	C-Mn	1% Cr	C-Mn HT	Low Alloy HT	Low Alloy HT
Chemistry						R260+up to 0.15%Cr	R260+up to 0.15%Cr	R260+up to 0.15%Cr

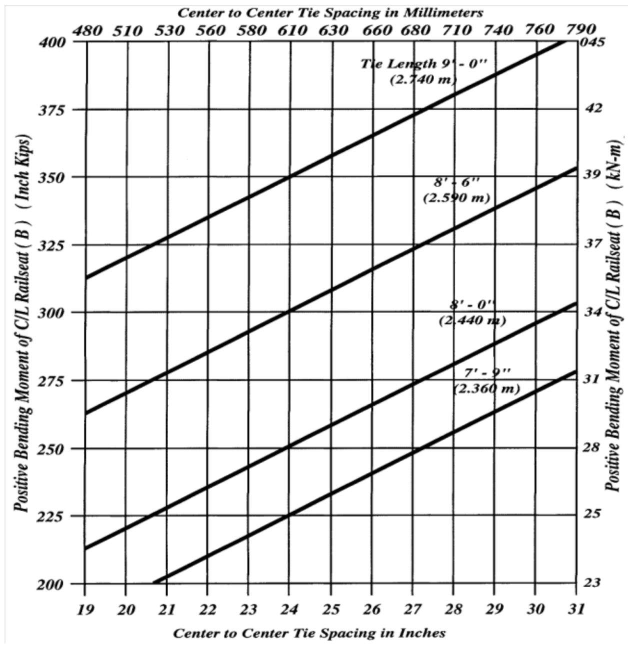
Note: The grades are designated according to their minimum hardness

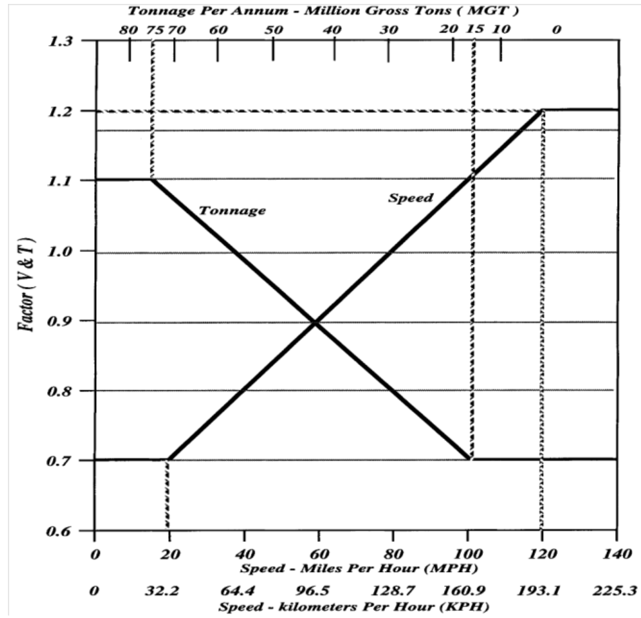




(a) (b)

Figure C-4: Estimated distribution factor of loads (a) AREMA and (b) Australia standards [AREMA Sec. 4.1.2.3, 2009 and AS 1085.24, 2003]





(a) (b)

Figure C-5: (a) Unfactored Bending Moment at Centerline of Rail Seat and (b) Tonnage and Speed Factors [AREMA Sec. 4.4.1, 2009]

Table C-3: Typical CBR ranges for different soil groups [ERA, 2002]

General Soil Type		USC Soil Type	CBR Range
Coarse-grained soils	gravel and gravelly soils	GW	40 - 80
		GP	30 - 60
		GM	20 - 60
		GC	20 - 40
	Sand and sandy soils	SW	20 - 40
		SP	10 - 40
		SM	10 - 40
		SC	5 - 20
Fine-grained soils	Silts and clays LL < 50	ML	15 or less
		CL, LL < 50%	15 or less
		OL	5 or less
	Silts and clays LL > 50	MH	10 or less
		CH, LL > 50%	15 or less
		OH	5 or less

Where:

Primary Letter	Secondary Letter
G: Gravel	W: well-graded
S: Sand	P: poorly graded
M: Silt	M: with non-plastic fines
C: Clay	C: with plastic fines
O: Organic soil	L: of low plasticity
	I: of medium plasticity
	H: of high plasticity

Table C-4: Material property of concrete and prestressing tendon with strength of 48 Mpa and s-250 respectively

Properties	Symbol	US system
Concrete		
Concrete strength	f'_c	7000 psi
Concrete strength at transfer	f'_{ci}	4500 psi *
Elastic Modulus	E_c	$57000\ (f'_c)^{0.5}$
Elastic Modulus at transfer	E_{ci}	$57000\ (f'_{ci})^{0.5}$
Extreme fiber stress in tension at transfer	σ_{ti}	$(-)0.25\ (f'_c)^{0.5} - (-)0.5\ (f'_c)^{0.5}$
Extreme fiber stress in tension at service	σ_{ts}	$(-)0.62\ (f'_c)^{0.5}$
Extreme fiber stress in comp at transfer	σ_{ci}	$0.6\ f'_{ci}$
Extreme fiber stress in comp at service	σ_{cs}	$0.45\ f'_{ci}$
Steel Tendon (s-250)		
Ultimate strength	f_{pu}	255 ksi
Yield strength	f_{pu}	230 ksi
Elastic modulus	E_s	28500 ksi

Note: (*) For concrete strength of 7000 psi, if the maturity equation is not given, take strength of 4500 psi at transfer

Appendix D: Analysis of Some Track Elements

Part-1: Track Geometry Elements

Table: D-1. Example of optimizing track cant and horizontal curve radius for mixed traffic line of maximum passenger train speed of 140, 160, 180, 200 and 240 km/hr and minimum freight train speed of 60, 80, 100 and 120 km/hr

V_{min}	h_d	h_e	h_t(optimized) (mm)					R_{min} (optimized) (m)				
			V_{max} (km/hr)					V_{max} (km/hr)				
			140	160	180	200	240	140	160	180	200	240
60	70	25	46	41	37	34	31	1987	2733	3577	4521	6707
80		25	71	57	48	43	37	1640	2385	3229	4173	6360
100		25	124	86	67	57	45	1192	1938	2782	3726	5912
120		25	288	147	101	78	57	646	1391	2236	3180	5366
60	70	30	53	46	43	40	37	1888	2596	3398	4295	6372
80		30	78	63	55	49	43	1558	2266	3068	3965	6042
100		30	134	94	75	63	51	1133	1841	2643	3540	5617
120		30	307	159	110	86	63	614	1322	2124	3021	5098
60	70	40	65	58	54	51	47	1716	2360	3089	3905	5793
80		40	93	77	67	61	54	1416	2060	2789	3604	5492
100		40	155	111	89	77	63	1030	1673	2403	3218	5106
120		40	345	181	128	102	77	558	1201	1931	2746	4634
60	70	50	77	70	65	62	58	1573	2163	2832	3579	5310
80		50	108	90	80	73	65	1298	1888	2557	3304	5035
100		50	175	127	104	90	75	944	1534	2203	2950	4681
120		50	382	204	146	118	90	511	1101	1770	2517	4248
60	70	55	83	75	71	67	63	1510	2077	2719	3436	5098
80		55	116	97	86	79	71	1246	1812	2454	3172	4833
100		55	185	135	111	97	81	906	1473	2115	2832	4493
120		55	401	216	155	125	97	491	1057	1699	2417	4078
60	70	65	95	87	82	78	74	1399	1923	2517	3182	4720
80		65	130	110	98	91	82	1154	1678	2273	2937	4475
100		65	206	152	125	110	93	839	1364	1958	2622	4161
120		65	439	239	173	141	110	455	979	1573	2238	3776
60	70	70	102	93	88	84	79	1349	1854	2427	3068	4551
80		70	138	117	104	97	88	1113	1618	2191	2832	4315
100		70	216	160	133	117	99	809	1315	1888	2529	4012
120		70	458	250	182	149	117	438	944	1517	2158	3641
60	70	85	120	110	104	100	95	1218	1675	2193	2771	4111

80		85	160	137	123	115	104	1005	1462	1979	2558	3898
100		85	246	184	154	137	118	731	1188	1705	2284	3624
120		85	514	284	209	172	137	396	853	1370	1949	3289
60	90	25	51	44	39	36	33	1642	2257	2955	3735	5541
80		25	81	63	53	47	39	1354	1970	2668	3448	5254
100		25	145	99	76	63	49	985	1601	2298	3078	4884
120		25	343	173	117	90	63	534	1149	1847	2627	4433
60	90	30	57	50	45	42	38	1573	2163	2832	3579	5310
80		30	88	70	60	53	45	1298	1888	2557	3304	5035
100		30	155	107	84	70	55	944	1534	2203	2950	4681
120		30	362	184	126	98	70	511	1101	1770	2517	4248
60	90	40	69	61	56	53	49	1452	1997	2614	3304	4902
80		40	103	83	72	65	56	1198	1743	2360	3050	4647
100		40	175	123	98	83	67	871	1416	2033	2723	4321
120		40	400	207	144	113	83	472	1017	1634	2324	3921
60	90	50	82	73	68	64	59	1349	1854	2427	3068	4551
80		50	118	97	84	77	68	1113	1618	2191	2832	4315
100		50	196	140	113	97	79	809	1315	1888	2529	4012
120		50	438	230	162	129	97	438	944	1517	2158	3641
60	90	55	88	79	73	69	65	1302	1790	2344	2962	4394
80		55	125	103	91	83	73	1074	1562	2116	2734	4167
100		55	206	148	120	103	85	781	1270	1823	2441	3874
120		55	457	241	171	137	103	423	911	1465	2083	3516
60	90	65	100	90	84	80	75	1218	1675	2193	2771	4111
80		65	140	117	103	95	84	1005	1462	1979	2558	3898
100		65	226	164	134	117	98	731	1188	1705	2284	3624
120		65	494	264	189	152	117	396	853	1370	1949	3289
60	90	70	106	96	90	86	81	1180	1623	2124	2685	3983
80		70	148	123	109	100	90	974	1416	1918	2478	3776
100		70	237	173	141	123	104	708	1151	1652	2213	3511
120		70	513	276	198	160	123	384	826	1328	1888	3186
60	90	85	124	114	107	102	97	1079	1483	1942	2454	3641
80		85	170	143	128	118	107	890	1295	1753	2266	3452
100		85	267	197	163	143	122	647	1052	1510	2023	3210
120		85	570	310	225	183	143	351	755	1214	1726	2913
60	110	25	55	47	42	38	34	1399	1923	2517	3182	4720
80		25	90	70	58	51	42	1154	1678	2273	2937	4475
100		25	166	112	85	70	53	839	1364	1958	2622	4161
120		25	399	199	133	101	70	455	979	1573	2238	3776

60	110	30	62	53	48	44	39	1349	1854	2427	3068	4551
80		30	98	77	64	57	48	1113	1618	2191	2832	4315
100		30	176	120	93	77	59	809	1315	1888	2529	4012
120		30	418	210	142	109	77	438	944	1517	2158	3641
60	110	40	74	65	59	55	50	1259	1731	2266	2863	4248
80		40	113	90	77	69	59	1038	1510	2045	2643	4028
100		40	196	136	107	90	72	755	1227	1762	2360	3745
120		40	455	233	160	124	90	409	881	1416	2014	3398
60	110	50	86	76	70	66	61	1180	1623	2124	2685	3983
80		50	128	103	89	80	70	974	1416	1918	2478	3776
100		50	217	153	121	103	84	708	1151	1652	2213	3511
120		50	493	256	178	140	103	384	826	1328	1888	3186
60	110	55	92	82	76	71	66	1144	1573	2060	2603	3862
80		55	135	110	96	86	76	944	1373	1859	2403	3662
100		55	227	161	129	110	90	687	1116	1602	2145	3404
120		55	512	267	187	148	110	372	801	1287	1831	3089
60	110	65	104	94	87	82	77	1079	1483	1942	2454	3641
80		65	150	123	108	98	87	890	1295	1753	2266	3452
100		65	247	177	143	123	102	647	1052	1510	2023	3210
120		65	550	290	205	163	123	351	755	1214	1726	2913
60	110	70	111	99	93	88	82	1049	1442	1888	2386	3540
80		70	157	130	114	104	93	865	1259	1704	2203	3356
100		70	258	185	150	130	108	629	1023	1468	1967	3120
120		70	568	301	214	171	130	341	734	1180	1678	2832
60	110	85	129	117	109	104	98	968	1331	1743	2203	3268
80		85	180	150	133	122	109	799	1162	1573	2033	3098
100		85	288	210	172	150	126	581	944	1355	1815	2880
120		85	625	336	241	195	150	315	678	1089	1549	2614

Table: D-2. Example of transition curve length [m] at different values of cant deficiency variation and rate of cant deficiency, $L_t \geq \frac{V_{max}}{3.6} * \Delta h_d \left(\frac{dh_d}{dt}\right)^{-1}_{lim}$

Δh_d (mm)	dh_d/dt (mm/s)	Speed, V_{max} (km/h)									
		120	140	160	180	200	220	240	260	280	300
70	30	78	91	104	117	130	143	156	169	181	194
	40	38	68	78	88	97	107	117	126	136	146
	50	30	54	62	70	78	86	93	101	109	117
	60	25	45	52	58	65	71	78	84	91	97
	70	22	39	44	50	56	61	67	72	78	83
100	30	111	130	148	167	185	204	222	241	259	278
	40	83	97	111	125	139	153	167	181	194	208
	50	67	78	89	100	111	122	133	144	156	167
	60	56	65	74	83	93	102	111	120	130	139
	70	48	56	63	71	79	87	95	103	111	119
130	30	144	169	193	217	241	265	289	313	337	361
	40	108	126	144	163	181	199	217	235	253	271
	50	87	101	116	130	144	159	173	188	202	217
	60	72	84	96	108	120	132	144	156	169	181
	70	62	72	83	93	103	113	124	134	144	155
150	30	167	194	222	250	278	306	333	361	389	417
	40	125	146	167	188	208	229	250	271	292	313
	50	100	117	133	150	167	183	200	217	233	250
	60	83	97	111	125	139	153	167	181	194	208
	70	71	83	95	107	119	131	143	155	167	179
180	30	200	233	267	300	333	367	400	433	467	500
	40	150	175	200	225	250	275	300	325	350	375
	50	120	140	160	180	200	220	240	260	280	300
	60	100	117	133	150	167	183	200	217	233	250
	70	86	100	114	129	143	157	171	186	200	214
200	30	222	259	296	333	370	407	444	481	519	556
	40	167	194	222	250	278	306	333	361	389	417
	50	133	156	178	200	222	244	267	289	311	333
	60	111	130	148	167	185	204	222	241	259	278
	70	95	111	127	143	159	175	190	206	222	238

Table: D-3. Example of transition curve length [m] at different values of cant variation and cant gradient, $L_t \geq \Delta h_t \left(\frac{dh_t}{dx}\right)_{lim}^{-1}$

Δh_t(mm)	$\left(\frac{dh_t}{dx}\right)^{-1}$(mm/mm)	L_t (m)
80	450	36
	500	40
	550	44
	600	48
	650	52
	700	56
	750	60
	800	64
	850	68
	900	72
	1000	80
	1100	88
	1200	96
	1300	104
	1400	112
	1500	120
100	450	45
	500	50
	550	55
	600	60
	650	65
	700	70
	750	75
	800	80
	850	85
	900	90
	1000	100
	1100	110
	1200	120
	1300	130
	1400	140
	1500	150

Δh_t(mm)	$\left(\frac{dh_t}{dx}\right)^{-1}$(mm/mm)	L_t (m)
130	450	58.5
	500	65
	550	71.5
	600	78
	650	84.5
	700	91
	750	97.5
	800	104
	850	110.5
	900	117
	1000	130
	1100	143
	1200	156
	1300	169
	1400	182
	1500	195
150	450	67.5
	500	75
	550	82.5
	600	90
	650	97.5
	700	105
	750	112.5
	800	120
	850	127.5
	900	135
	1000	150
	1100	165
	1200	180
	1300	195
	1400	210
	1500	225

Where: $\left(\frac{dh_t}{dx}\right)^{-1}$ is inverse of cant gradient

Formulation of Horizontal and Vertical curves elements

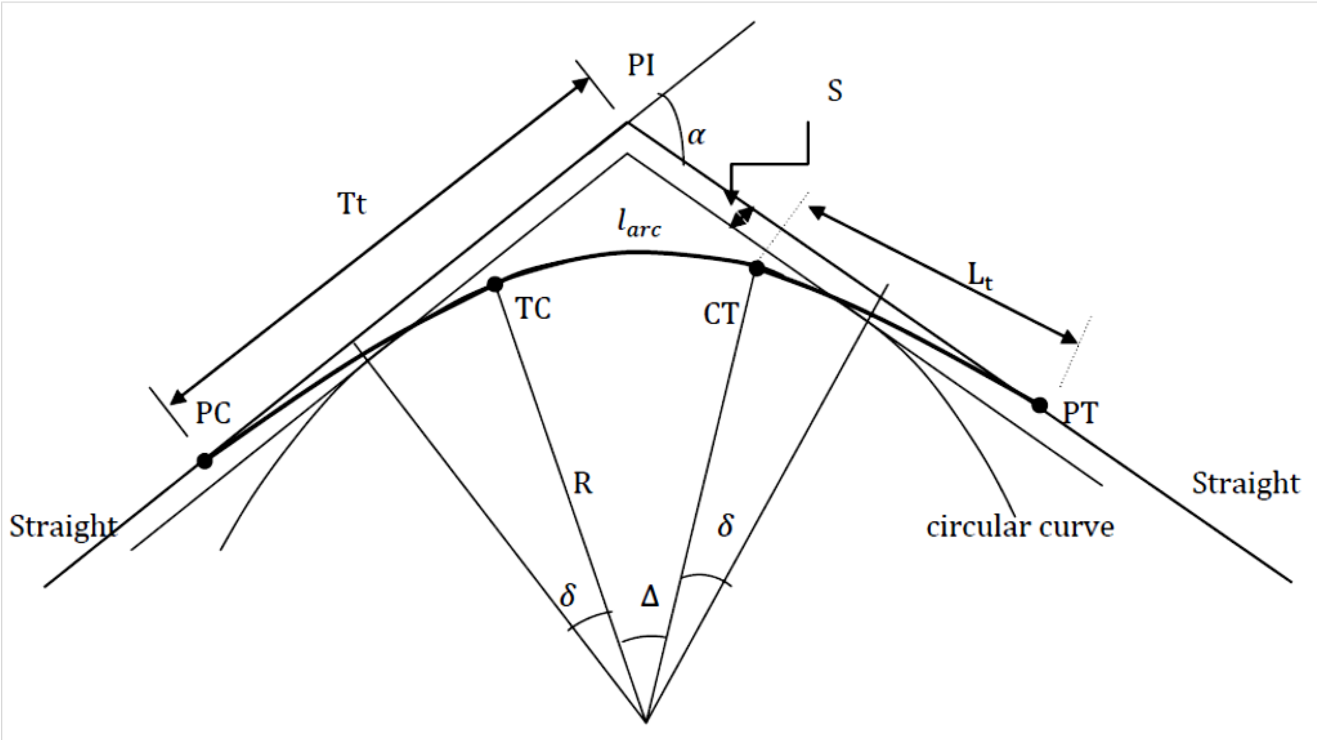


Figure D-1: Horizontal curve with transition curve elements

Terminology and Definition

R = Radius of circular curve

S = Shift distance is the offset from the tangent to the point on the circle at which the radius is normal to it, and approximated to $S = L_t^2/24R$

α = Deflection Angle between the two tangents

$\alpha = \Delta + 2\delta$

Δ = Central angle of the circular curve

PI = Point of intersection

PC = Point of curvature (start of transition curve)

PT = Point of tangent (end of transition curve)

TC = change from transition curve to circular curve

CT = change from circular curve to transition

L_t = Transition curve length

Tt = Length between PI and PC

l_{arc} = Arc length of circular curve between point TC and CT

Formula of different curve elements

Tangent length, $T = (R + S) \tan(\alpha/2)$

The distance from the PI to the PC (point of curvature or start of transition)

Tt = T + ($L_t - R \sin\Delta$), Approximate value of ($L_t - R \sin\Delta$) $= L_t/2$. Therefore,

Tt = (R + S) tan ($\alpha/2$) + $L_t/2$

Arch length, $l_{arc} = R * \alpha - L_t$, where α in radian (i.e., $\alpha^\circ * \pi/180$)

Chainages

Chainage PC = Chainage PI – Tt

Chainage TC = Chainage PC + L_t

Chainage CT = Chainage TC + l_{arc}

Chainage PT = Chainage CT + L_t

Vertical Curve

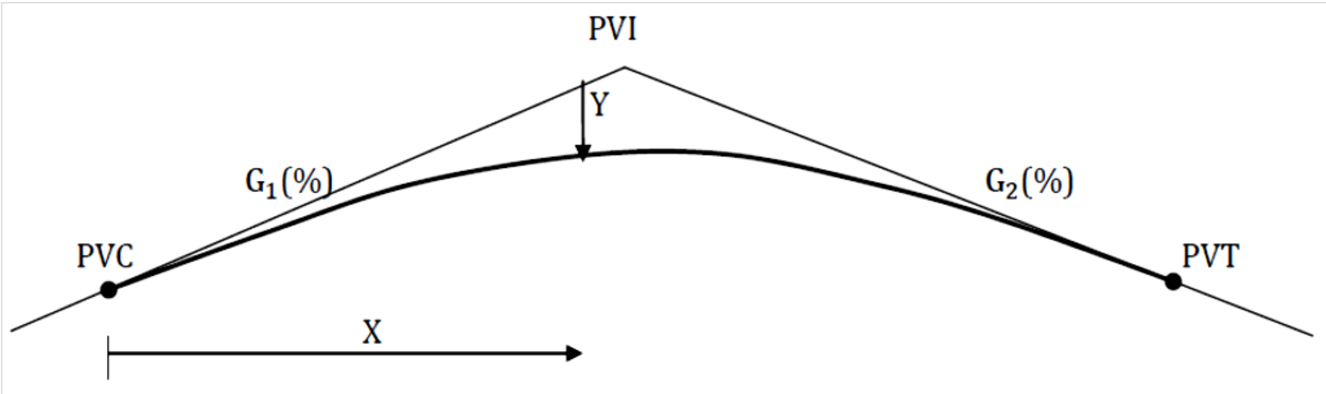


Figure D-2: Vertical curve (e.g. summit curve) elements

Terminology and Definition

PVI = Point of vertical intersection of tangent lines

PVC = Point of vertical curvature

PVT = Point of vertical tangency

L = Length of curve

G_1 = initial roadway grade in percent

G_2 = final roadway grade in percent

A = absolute value of difference in grades

Y = Offset – vertical distance from initial tangent to the curve at distance x from PVC

Offset,

$$Y = \frac{A}{200L} x^2$$

Elevation of tangents and stations along the vertical curve (for example of figure)

Elevation of PVC

$$PVC_{EL} = PVI_{EL} + (-1 * G_1 * L/2)$$

Elevation of PVT

$$PVT_{EL} = PVI_{EL} + (G_2 * L/2)$$

Elevation of station along the back tangent

$$\text{Elevetion of station} = PVC_{EL} + (G_1 * \text{distance from PVC})$$

Elevation of the stations along the forward tangent

$$\text{Elevation of station} = PVT_{EL} + (G_2 * \text{distance from PVT})$$

Part-2: Track Structures

D.2.1. Optimization of Sleeper Dimension

Optimization can be done, for example, with the following assumption of

- 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.

From equation of average pressure equation, minimum average base width- $b = \frac{q_r}{P_a \times (l-g)}$; for $q_r = 3 \times DF \times 90 = 270DF$, $P_a = 450kPa$ (allowable pressure) and g = 1.5 m tabulated below

Table D-4: 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

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.								

The volume of concrete sleeper (V) can be calculated as:

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

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

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

The total cost of sleepers per kilometer can be calculated as

$$\text{Total cost} = V \times N \times 6000 \text{ birr/km}$$

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

Table D-5: Optimization of sleeper dimension and sleeper spacing for a wheel load of 90 KN that produce 444 kPa pressure at bottom of sleeper

Spacing (mm)	l (mm)	b (mm)**	T (mm)	h (mm)	V (m^3)	N (no/km)	Total cost (10^6)	f	Total Cost (10^6)*
520	2.4	311	175	215	0.125	1923	1.45	1	1.45
520	2.5	280	158	215	0.118	1923	1.36	1	1.36
520	2.55	266	150	215	0.114	1923	1.31	1	1.31
520	2.6	254	143	215	0.111	1923	1.28	1.02	1.31
520	2.65	244	137	215	0.109	1923	1.25	1.01	1.27
550	2.4	325	183	215	0.131	1818	1.43	1.02	1.46
550	2.5	292	164	215	0.123	1818	1.34	1.02	1.36
550	2.55	278	156	215	0.119	1818	1.30	1.02	1.33
550	2.6	265	149	215	0.116	1818	1.26	1.02	1.29
550	2.65	254	143	215	0.113	1818	1.23	1.02	1.26
550	2.7	243	137	215	0.110	1818	1.20	1.02	1.23
580	2.5	304	171	215	0.128	1724	1.32	1.04	1.37
580	2.55	290	163	215	0.124	1724	1.29	1.04	1.34
580	2.6	276	155	215	0.121	1724	1.25	1.04	1.30
580	2.65	265	149	215	0.118	1724	1.22	1.04	1.27
580	2.7	253	142	215	0.115	1724	1.19	1.04	1.23
610	2.5	316	178	215	0.133	1639	1.31	1.06	1.38
610	2.55	302	170	215	0.129	1639	1.27	1.06	1.35
610	2.6	288	162	215	0.126	1639	1.24	1.06	1.31
610	2.65	275	155	215	0.122	1639	1.20	1.06	1.28
610	2.7	264	149	215	0.120	1639	1.18	1.06	1.25
640	2.5	325	183	215	0.136	1563	1.28	1.08	1.38
640	2.55	310	174	215	0.133	1563	1.24	1.08	1.34
640	2.6	295	166	215	0.129	1563	1.21	1.08	1.30
640	2.65	282	159	215	0.126	1563	1.18	1.08	1.27
640	2.7	271	152	215	0.123	1563	1.15	1.08	1.24
670	2.55	318	179	215	0.136	1493	1.22	1.1	1.34
670	2.6	304	171	215	0.133	1493	1.19	1.1	1.31
670	2.65	291	164	215	0.130	1493	1.16	1.1	1.28

670	2.7	279	157	215	0.127	1493	1.13	1.1	1.25
700	2.55	330	186	215	0.141	1429	1.21	1.12	1.36
700	2.6	315	177	215	0.138	1429	1.18	1.12	1.32
700	2.65	302	170	215	0.134	1429	1.15	1.12	1.29
700	2.7	289	163	215	0.131	1429	1.12	1.12	1.26
730	2.6	320	180	215	0.140	1370	1.15	1.15	1.32
730	2.65	307	173	215	0.137	1370	1.12	1.15	1.29
730	2.7	294	165	215	0.133	1370	1.10	1.15	1.26

Note: * The total cost of sleeper per kilometer track length that factored to be quality.
** The width of sleeper at rail seat is greater than the calculated average width of sleeper to increase bearing capacity.
Where: f-quality increment factor

D.2.2. Sleeper Design Procedure

Profile of PSC sleeper cross section

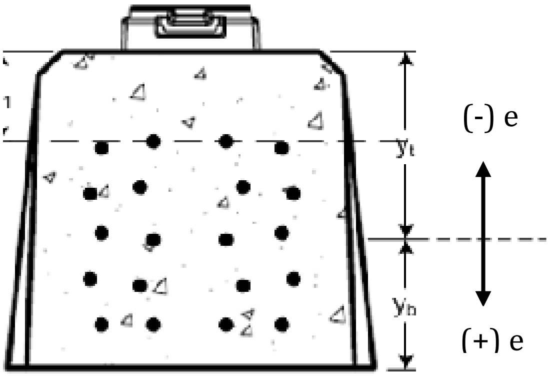


Figure: Number of prestressing steel and prestressing eccentricity in typical rail seat section of PSC sleeper

The prestressing eccentricity is the difference between the uncracked neutral axis and center of steel tendon. The sign convention used with positive for a prestressing centroid below the uncracked neutral axis and negative for above the uncracked neutral axis.

Permissible stresses in prestressing steel

The tensile stress in prestressing steel shall not exceed in the following cases:

a. Due to prestressing steel jacking force:

$$f_{pj} < 0.94\, f_{py} \text{ and } f_{pj} < 0.8\, f_{pu}$$

Where: The prestressing steel stress after jacking, $f_{pj} = \frac{F_{pj}}{A_{tendon}}$; the jacking force, $F_{pj} = 7000$ lb (manufacturer standards) and the ultimate capacity is 8800 lb force.

b. Immediately after prestress transfer (include anchorage, short term relaxation and elastic shortening loses)

$$f_{pi} < 0.82\, f_{py} \text{ and } f_{pi} < 0.74\, f_{pu}$$

Where: f_{pi} –computed by directly measuring forces at rail seat (F_{pi-RS}) and at center section (F_{pi-C}) by manufacturer

$$f_{pi-RS} = \frac{F_{pi-RS}}{A_{ps}}$$

$$f_{pi-C} = \frac{F_{pi-C}}{A_{ps}}$$

c. After all losses

Assuming a lump sum loses in percent for the time dependent loses at the rail seat and rail center section, the final prestressing stress (f_{pe}) in the prestressing steel can be calculated.

$$f_{pe} = f_{pj} - f_{pl}$$

Where: f_{pl} –is time dependent loses of stress and it can be assumed. But it can be vary at rail seat and center section.

Allowable concrete stresses at transfer

Stresses at transfer are only due to self-weight and prestressing force shall be less than the extreme fiber stress in compression at transfer at rail seat.

$$\sigma_{top} = \frac{F_i}{A_{c-RS}} - \frac{F_i.\, e_{RS}.\, c_{t-RS}}{I_{x-RS}} + M_0 \times \frac{c_{t-RS}}{I_{x-RS}} < \sigma_{ci} = 0.6\, f'_{ci}$$

$$\sigma_{bot} = \frac{F_i}{A_{c-RS}} + \frac{F_i.\, e_{RS}.\, c_{b-RS}}{I_{x-RS}} - M_0 \times \frac{c_{b-RS}}{I_{x-RS}} < \sigma_{ci} = 0.6\, f'_{ci}$$

$$F_i = f_{pi}\, A_{ps} - \text{Prestressing force}$$

Where:

- f_{pi} –prestressing stress at transfer;
- A_{ps} –area of prestressing tendon;
- e_{RS} –eccentricity of prestressing tendon;
- c_{b-RS} –distance from bottom to neutral axis (for bottom stress);
- c_{t-RS} –distance from top to neutral axis (for top stress);
- I_{x-RS} – moment of inertia;
- M_0 –moment due to self weight of PSC sleeper;
- A_{c-RS} –area of concrete cross section; and
- σ_{ci} – Extreme fiber stress in compression at transfer.

Similarly stresses at center section are also computed as above with similar fashion by substituting parameters of center section.

Moment Capacity Analysis based on Allowable Concrete Stresses

Concrete railroad tie failure and the corresponding capacity as defined by AREMA are dictated by cracking to the outer most layer of prestressing. Therefore the moments required to initiate cracking at the critical sections of the rail seat and center are of interest. To avoid working with cracked sections and determining the rate of crack propagation, the tie will assume to be failed when the stresses in the tie reach the allowable stresses at the extreme compression and tension fibers. For this capacity analysis the final prestressing force at rail seat (F_{RS}) and center section (F_C) of PSC sleeper can be calculated using the final prestressing stresses (f_{pe}) that including all prestress losses.

$$F_{RS} = f_{pe-RS}\, A_{ps}$$

$$F_C = f_{pe-C}\, A_{ps}$$

Theoretical Cracking Moment

Maximum positive moment based on allowable concrete stresses at the rail seat section

I. Positive cracking moment (top stress)

$$\sigma_{cs} = \frac{F_{RS}}{A_{c-RS}} - \frac{F_{RS}.\, e_{RS}.\, c_{t-RS}}{I_{x-RS}} + M_c \times \frac{c_{t-RS}}{I_{x-RS}}, \qquad M_c = \cdots$$

II. Positive cracking moment (bottom stress)

$$\sigma_{ts} = \frac{F_{RS}}{A_{c-RS}} + \frac{F_{RS}.\, e_{RS}.\, c_{b-RS}}{I_{x-RS}} - M_c \times \frac{c_{b-RS}}{I_{x-RS}}, \qquad M_c = \cdots$$

Where:

$\sigma_{cs} = 0.45\, f'_c$ – Allowable concrete compressive stress at time of effective prestress

σ_{ts} = Allowable concrete tensile stress at time of effective prestress

M_c = Cracking moment

The maximum positive value at rail seat controlled by allowable tension stress shall be selected.

Maximum negative moment based on allowable concrete stresses at the rail seat section

I. Negative cracking moment (top stress)

$$\sigma_{ts} = \frac{F_{RS}}{A_{c-RS}} - \frac{F_{RS}.\, e_{RS}.\, c_{t-RS}}{I_{x-RS}} + M_c \times \frac{c_{t-RS}}{I_{x-RS}}, \qquad M_c = \cdots$$

II. Negative cracking moment (bottom stress)

$$\sigma_{cs} = \frac{F_{RS}}{A_{c-RS}} + \frac{F_{RS}.\, e_{RS}.\, c_{b-RS}}{I_{x-RS}} - M_c \times \frac{c_{b-RS}}{I_{x-RS}}, \quad M_c = \cdots$$

The maximum negative value at rail seat controlled by allowable tension stress shall be selected.

Note: The maximum positive and negative moment based on allowable concrete stresses at the center section can be computed with similar fashion as above with different value of the parameters.

Experimental Flexural Test

Experimental test results were provided by the tie manufacturer in the form of applied loads corresponding to failure. Then the corresponding value of cracking moment at critical section of PSC sleeper can be calculated, theoretical computation, using this load capacity at different test configuration.

Finally the flexural capacity at critical section of the PSC sleeper based on theoretical and experimental result shall be compared. According to these two methods of flexural capacity calculation, the cracking moment must be greater than the calculated design moment.

Part-3- Safety- SMS and ESM Safety Documents

1. Safety management (SMS) documents may include [Noh et al., 2010]:
    a) Contents of safety policy
    b) Establishment and procedure of safety targets
    c) Observance of safety law
    d) Risk assessment and management measures
    e) Education and qualification management of railway personnel
    f) Management of safety information
    g) Documentation of safety management
    h) Investigation and reports of accidents
    i) Emergency measures plans
    j) In-house audits and evaluation for improvement of SMS
3. Engineering safety management (ESM) documents include [Noh et al., 2010]
    a) Preliminary safety plan documents
    b) Preliminary hazard identification documents
    c) Risk assessment documents
    d) Safety case documents
    e) Safety requirement description documents

DECLARATION

I hereby declare that the work which is being presented in this thesis entitled **"STANDARDIZATION OF GUIDELINES FOR RAILWAY TRACK INFRASTRUCTURE SUBSYSTEM FOR RAILWAY SYSTEM OF ETHIOPIA"** is original work of my own, has not been presented for a degree in any other university; and that all sources of material used for the thesis have been duly acknowledged.

Yeserah Gebeyehu Asegie	--------------------	--------------------
(Candidate)	Signature	Date

Submitted to: **Addis Ababa University, Addis Ababa Institute of Technology**