Railway Transportation Systems

Design, Construction and Operation

CHRISTOS N. PYRGIDIS

CRC Press
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Aristotle University of Thessaloniki, Greece

CRC Press
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Boca Raton London New York

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business
A SPON PRESS BOOK

CRC Press
Taylor & Francis Group
6000 Broken Sound Parkway NW, Suite 300
Boca Raton, FL 33487-2742

Version Date: 20160104

International Standard Book Number-13: 978-1-4822-6216-2 (eBook - PDF)

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To my wife, Maria and my son, Nikos

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Preface

> The development of railway technology reached an early peak and then was found to be in question. During the past decade it has not only managed to rise again, but also be on the cutting edge of technology in many countries.

The term 'railway transportation systems' includes all means of transport whose rolling systems involve at least one iron component (steel wheels on rails or rubber-tired wheels on steel guideway). Because of this, this book examines only transport systems that have this particular characteristic in common.

This book presents a comprehensive overview of passenger and freight railway transport systems, from design through to construction and operation. It covers the range of railway passenger systems, from conventional and high-speed interurban systems through suburban, regional, urban and rail transport systems for steep gradients. Moreover, it thoroughly covers freight railway systems transporting conventional loads, heavy loads and dangerous goods. For each system it provides a definition, a brief overview of its evolution and examples of good practice, the main design, construction and operational characteristics, the preconditions for its selection and the steps required to verify the feasibility of its implementation.

The book provides a general overview of issues related to safety, interfaces with the environment, cutting-edge technologies and finally the techniques that govern the stability and guidance of railway vehicles on track.

It incorporates the author's 25 years of involvement in teaching, research and experience in railway engineering.

Until recently, knowledge of railway technology was shared only among railway organisations. Many of the organisations' executives changed job positions in order to broaden their vision and knowledge. In recent years, an increasing number of people have become involved in the field of rail transport worldwide (engineers, consultants, manufacturers, transport companies, etc.).

This book provides additional information for those interested in learning about railway transportation systems. It can be used as a decision-making tool for both designers and operators of railway systems. In addition, it attempts to educate young railway engineers to enable them to deal with rail issues that may be assigned to them during the course of their careers.

All the data recorded and analysed in this book relate to the end of year 2014. The raw data were obtained per country, per city and per line, from various available sources and were cross-checked.

Professor Christos N. Pyrgidis
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Acknowledgements

I thank everyone who has helped me complete this work.

Specifically, I wish to thank:

- My colleagues: Constantina Anastasiadou, Nikolaos Asmeniadis, Alexandros Panagiotopoulos, Rafael Katkadigkas, civil engineers, Dr. Evangelos Christogiannis, Dr. Alexandros Deloukas, Dr. Nikolaos Demiridis, Dr. Nikolaos Gavanas, Dr. Anastasios Lambropoulos, Dr. Georgios Leoutsakos, Professor Antonios Stathopoulos, Professor François Lacote. All of them agreed with pleasure to read through chapters of my book and provided useful comments.
- Ifigeneia Balampekou, transport engineer, for assisting with the translation and text editing.
- Artemis Klonos, professional photographer, who provided most of the pictures in this book.
- Ntia Regka, architect, for editing the figures.
- My dear friend Alexandros Kolokotronis, who helped in the selection and design of the book's cover.
- All my graduate and postgraduate students, who contributed in different ways to the final result.

I acknowledge all organisations, institutions, private companies and individuals for providing and permitting me to use tables, figures and photographs. Specifically, I thank: edilon)(sedra, Siemens, SNCF, SNCF Médiathèque, Posco ICT Co. Ltd, DCC Doppelmayr Cable Car, Innsbrucker Nordkettenbahnen Betriebs GmbH, Voestalpine Nortrak Ltd., Talgo, Vossloh Fastening Systems GmbH, Mermecgroup, BONATRANS GROUP a.s., Peer plus, Struktonrail bv, ANSALDO STS, Bombardier Transport, Alstom Transportation, UIC, ABB, Railway Gazette International, RATP, UITP, LEA+ELLIOTT Inc., Eyrolles Editions, MRT-Productions, Transportation Safety Board of Canada, CERTU, LEITNER AG/SpA, U.S. Department of Transportation Federal Highway Administration, Urbanaut, Hitachi Ltd, Professor C. Hass-Klau, Ev. Krussev, K. Pedersen, A. Takahashi, Br. Finn, Dr. K. Giannakos, L. Zlateva, Professor S. Iwnicki, Professor V. Profillidis, Associate Professor K. Vogiatzis, Professor Coenraad Esveld, D. Oikonomidis, Ben Betgen, Harald Loy and S. Dale.

Last but not least, I thank Taylor & Francis Group for editing and publishing this book and for the excellent collaboration throughout the process.

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Author

Christos N. Pyrgidis was born January 1957 in Thessaloniki, Greece. He is a professor of railway engineering in the Civil Engineering Department, Aristotle University of Thessaloniki (AUTh), Greece. He earned a diploma in civil engineering (AUTh, 1981). He specialised for 5 years at the Ecole Nationale des Ponts et Chaussées (ENPC) Paris, France in transportation infrastructure (1984–1985: CES 'Infrastructure et Transports'); transport economics (1985–1986: DEA 'transport') and railway engineering (1986–1989: PhD in 'transport'). He has been conducting courses in the Civil Engineering Department of AUTh on railways, design and management of railway transportation systems (master's level) and construction methods of transportation technical structures (master's level). He has participated in many research railway projects and studies and has worked as consultant to several railway organisations. From 2004 to 2007, he was the Greek representative to the Administrative Board of the European Railway Agency (ERA). Since July 2014, he has been a member of the ERRAC Academia Permanent Advisory Group. Professor Pyrgidis is a member of the Scientific Committee of SHIF2RAIL Joint Undertaking (S2R JU). He has published more than 100 articles in scientific journals and international congresses. He is the author of another book entitled *Railway Transportation Systems: Infrastructure, Rolling Stock, Exploitation* (in Greek).

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Symbols and Abbreviations

A	track category in accordance with UIC (based on the permitted axle load)
A_r	rail cross section
A_p	bogies pivot centre
A_w	parameter depending on the characteristics of the rolling stock and representing the rolling resistance
AC	alternative current
AFC	automatic fare collection system
AGT	automated guideway transit system
APMs	automated people movers
APS	Alimentation Par le Sol – Ground power supply system for tramways
APT	advanced passenger train
ATO	automatic train operation
ATP	automatic train protection system
ATS	automatic train supervision system
a_b	parameter that depends on the classification of the track in the UIC classes
a_d	lateral distance of the noise barrier from the track centre
2a	bogie wheelbase
B	track category in accordance with UIC (based on the permitted axle load)
B_r	rail's weight per metre
B_t	vehicle weight
B_{tr}	train's weight
B_{ty}, B_{tz}	lateral and vertical components of the vehicle weight–motion in curves
B_w	parameter depending on the characteristics of the rolling stock and representing the mechanical resistances
BODu	biochemical oxygen demand
BTU	British Thermal Unit
b_b	parameter depending on the sleepers' length and material
b_{cp}	width of a centre (island) platform
b_{em}	width needed for the installation of electrification masts
b_{lp}	width of a side platform
b_{sw}	width of separator (tramway corridors)
b1, b2	width of two intersected roads (1 and 2) (tramway network)
C	track category in accordance with UIC (based on the permitted axle load)

C_{hmin}	constructional height in the middle of the catenary opening
C_o, C'_o	track maintenance cost
C_{thr}	centre throw
C_v	transport capacity of a passenger train or vehicle
C_{vph}	transport capacity of a passenger train or vehicle in peak hours
C_w	parameter depending on the characteristics of the rolling stock and representing aerodynamic resistance
$\bar{C}_x, \bar{C}_y, \bar{C}_z$	damping coefficients of the secondary suspension dampers in the three directions, respectively
C_ρ	damping torsional torque
C_φ	damping coefficient
CBTC	communications-based train control systems
CCTV	closed-circuit television
CIM	Convention Internationale Marchandises par Chemin de Fer (contract of international carriage of goods by rail)
COTIF	Convention Transports Internationaux Ferroviaires (convention concerning international carriage by rail)
CWR	continuous welded rails
c_b	parameter that depends on the volume of the required track maintenance work
c_{ij}	coefficients of Kalker
c_{11}	longitudinal creep coefficient of Kalker
c_{22}	transversal creep coefficient of Kalker
c_{23}, c_{33}	spin coefficients of Kalker
D	track category in accordance with UIC (based on the permitted axle load)
D_o	minimum wheel diameter of trains running along the line
DC	direct current
DMU	diesel multiple unit
DRNT	design rail neutral temperature
DTO	driverless train operation
DVT	driving van trailer
d_b	parameter that depends on the maximum axle load
d_o	vibration displacement
d'_o	reference vibration displacement
2d	lateral distances between springs and dampers of the primary suspension
$2d_a$	back-to-back wheel distance (inside gauge)
E	steel elasticity modulus
E_b	parameter that depends on the quality category of the soil and the bearing capacity of the substructure
E_c	track cant excess
E_{cmax}	maximum track cant excess
E_d	total ground plan area of a depot
E_{thr}	end throw

EMU	electrical multiple unit
ERA	European Railway Agency
ESS	energy storage systems
EU	European Union
EXCA	EXceeded capacity
e_b	ballast thickness
e_{bt}	total thickness of ballast and sub-ballast layers
e_{sb}	sub-ballast thickness
2e	track gauge
$2e_a$	outer flange edge-to-edge distance (flange gauge)
$2e_o$	theoretical distance between the running surfaces of the right and the left wheels when centered ≈ distance between the vertical axis of symmetry of the two rails
F	guidance effort exerted from the wheel to the rail
F_{cf}	centrifugal force
F_{cfy}, F_{cfz}	lateral and vertical components of the vehicle's centrifugal force – motion in curves
F_{ij}	guidance forces exerted from the four wheels of a 2-axle bogie to the rails (i = 1,2 front and rear wheelset, respectively; j = 1,2 left and right wheel, respectively, in the direction of movement)
F_{nc}	residual centrifugal force
F_t	traction effort developed on the driving wheel treads
F_{tr}	traction effort acting on the axles
f	frequency of oscillation
f_b	parameter that depends on the track design speed and the bearing capacity of the substructure
f_d	wheel flange width
G	geometrical centre of a railway wheelset
G′	centre of gravity of the car body
GoA	grade of automation (metro systems)
GPS	global positioning system
GRT	group rapid transit
g	gravity acceleration
g_{dv}	dynamic gauge width of tram vehicle
g_i	maximum track twist
g_{imax}	highest permitted value for track twist
H_R	lateral track resistance
H_w	cross wind force
h	height clearance under civil engineering structure
h_f	wheel flange height
h_{fc}	height of the catenary contact wire
h_{KB}	distance between the vehicle's centre of gravity and the rail rolling surface
h_o	track lifting after maintenance work
I	track cant deficiency

I_{max}	maximum track cant deficiency
I_1	isolation distance of wire-grounded structures
IRR	internal rate of return
i	track longitudinal gradient (or slope)
i_{max}	maximum track longitudinal gradient (or slope)
i_{min}	minimum track longitudinal gradient (or slope)
$j = 1, 2$	indicator relative to the two wheels of the same wheelset
K	factor of decrease of the aerodynamic load exerted to noise barriers
K′	factor of increase of the aerodynamic load exerted to noise barriers
K_b	angular stiffness of the link between the two wheelsets of the bogie (bogies with self-steering wheelsets)
K_{bt}	total longitudinal stiffness of the primary suspension system (bogies with self-steering wheelsets)
K_{dyn}	vertical dynamic stiffness
K_m	coefficient with values varying between 1.15 and 1.45
K_o	bogie–yaw dampers stiffness
K_{op}	operating cost
K_{st}	total lateral stiffness of the primary suspension (bogies with self-steering wheelsets)
K_{stat}	vertical static stiffness
K_t	coefficient that depends on the rolling conditions of the power vehicle axles on the track
K_x	longitudinal stiffness of the primary suspension (springs)
K_y	lateral stiffness of the primary suspension (springs)
$\bar{K}_z$	vertical stiffness of the secondary suspension (springs)
$K_ς$	lateral stiffness of the link between the two wheelsets of the bogie (bogies with self-steering wheelsets)
K_1	parameter depending on the shape of the 'nose' and the 'tail' of the train
K_2	parameter depending on the lateral external surface of a train
k	vertical track stiffness
L_{den}	day–evening–night equivalent noise level
L_{dn}	day–night equivalent noise level
$L_{eq,T}$	equivalent energy noise level
L_h	length over headstock
L_{kmin}	minimum allowed length for a transition curve
L_{max}	maximum noise level
L_{st}	distance between two successive stops
L_T	civil engineering structure width
L_{tr}	train's length
L_w	oscillation wave length (hunting of railway wheelset)
LC	locomotive
LCL	less-than-carload
LED	light emitting diode
LIM	linear induction motors

LRC	laser railhead cleaner
LRTs	light rail transport systems
LRV	light rail vehicles
LTL	less-than-truckload
l_A	expansion zone length of rail
l_o	initial rail length
l_T	civil engineering structure length
M	spin moment on wheels
M′	mass of one bogie
$\bar{M}$	car body mass
M_t	total mass of the vehicle
M_1, M_2	spin moment in the left and right wheels, respectively, in the direction of movement of a railway wheelset
MC	motor car
MU	multiple unit
m	mass of one railway wheelset (axle + wheels + axle-boxes)
N	temperature force
N′	reaction force in the wheel–rail contact surface
N_{ac}	acceleration force
N_{br}	braking force
NATM	New Austrian Tunneling Method
NPV	net present value
n_b	total number of bogies of a train formation
n_p	coefficient of the probability augmentation of the mean square value of standard deviations of vertical dynamic forces of a vehicle
n_s	number of intermediate stations/stops
OCS	overhead power supply (catenary) system
P_d	total number of passengers expected to be transported along a specific connection (passengers/hour/direction or daily-potential transport volume)
P'_d	passenger transport capacity of the system (passengers/hour/direction)
P_{dph}	total number of passengers expected to be transported along a specific route during the peak hours
P_{dyn}	transversal force due to vehicle oscillations
P_f	fishplate force
P_t	net or useful power of motors
P_{4w}	power consumed at the level of the four wheels of the bogie
PPV	peak particle velocity
PR	single railcar
PRT	personal rapid transit
PSD	platform screen doors
PSE	Paris–Sud–Est
PT	public means of transport
p	the perimeter that encloses the rolling stock laterally, up to rail level (rolling stock outline)

p_o	mean noise pressure
p'_o	the relative mean reference pressure
ppl	population of a city
Q	axle load
Q_d	design vertical wheel load
Q_{Do}	maximum passing axle load (wheels of diameter D_o)
Q_{dyn}	dynamic vertical wheel load
Q_{dyn1}	dynamic vertical wheel load due to the vehicle's sprung masses
Q_{dyn2}	dynamic vertical wheel load due to the vehicle's semi-sprung masses
Q_{dyn3}	dynamic vertical wheel load due to the vehicle's unsprung masses
Q_{dyn4}	dynamic vertical wheel load due to the oscillations of the elastic parts of the rail-sleeper fixing system
Q_H	quasi-static vertical wheel load
Q_{max}	maximum axle load or design vertical axle load of a railway infrastructure
Q_{nc}	vertical wheel load due to residual centrifugal force
Q_o	wheel load (=Q/2)
Q_t	total static vertical load of wheels (j = 1,2)
Q_w	vertical wheel load due to cross winds
Q_1, Q_2	total static vertical load of the left and right wheels, in the direction of movement of a railway axle
q	uniform load applied to noise barriers
q_o	vertical distance between the geometrical centre of the lateral surface of the car body and the rail rolling surface
q_r	flange cross-dimension (the horizontal distance between the intersection point of the joint geometric level with the flange face and the intersection point of a reference line at a distance of 2 mm from the flange tip with the flange face)
R	curvature radius of the wheel tread
R′	radius of curvature of the rolling surface of the rail head
R_c	radius of curvature in the horizontal alignment
R_{cmin}	minimum radius of curvature in the horizontal alignment
R_{co}	horizontal alignment radius as it derives from simulation models
R_s	sound-insulating capacity index of the construction material of noise barriers
R_v	radius of curvature in the vertical alignment
R_{vmin}	minimum radius of curvature in the vertical alignment
RID	Regulation International de Transport des Produits Dangereux par Chemin de fer (international carriage of dangerous goods by rail)
RLC	railway level crossing
RNT	rail neutral temperature
ROLA	ROllende LAndstraβe – rolling road or highway transport
r_o	rolling radius of the wheel in the central equilibrium position
r_1, r_2	rolling radius of the left and the right wheels in the direction of movement of a railway wheelset in case of lateral displacement from its central equilibrium position

$2r_o$	wheel diameter
S	route, link, or connection length
S_A, S_B, S_C, S_D, S_E	tramway corridor length for corridor categories A, B, C, D, E, respectively
S_c	affected cross section surface of the train
S_{max}	maximum route or link or connection length
S_{min}	minimum route or link or connection length
S_p	total gravitational force, restoration force or gravitational stiffness
S_{p1}, S_{p2}	gravitational forces exerted on the left and the right wheels when the wheelset is displaced from its central equilibrium position
S_u	useful cross section area of the tunnel
S_v, S_m	coefficients with values depending on the speed of passenger (with the highest speed) and freight (with the lowest speed) trains, respectively, running on the track
SEL	sound exposure level
SPD	suspended partical devices
SPL	sound pression level
STO	semi-automatic train operation
SNCF	Société Nationale des Chemin de Fer Français (national company of French railways)
SW	loading model in railway bridges (heavy loads)
SWL	single wagon load services
T	lateral creep force applied on the wheel
T_f	total daily traffic load
T_{fr}	friction forces between rails and sleepers and between sleepers and ballast
T_g	daily traffic load of freight trains
T_{ij}	lateral creep forces exerted to the four wheels of a 2-axle bogie (i = 1,2 front and rear wheelset, respectively, j = 1,2 left and right wheels, respectively, in the direction of movement)
T_m	average daily traffic load of trailer freight wagons
T_p	daily traffic load of passenger trains
T_{tm}	average daily traffic load of freight trains' power vehicles
T_{tv}	average daily traffic load of passenger trains' locomotives
T_V	average daily traffic load of trailer passenger cars
T_1, T_2	lateral creep forces exerted to left and right wheels (in the direction of movement) of a railway wheelset
TBM	tunnel boring machine
TC	trailer vehicle (car) of a train
TC′	trailer vehicle (car) of a railcar
TGV	Train Grande Vitesse (high-speed train [French technology])
TGV-A	TGV Atlantique (TGV Atlantic)
TL	train load services
TOFC	trailer of flat cars
TSIs	technical specifications interoperability

TT	single tramway track
TTROW	total tramway infrastructure right-of-way
TTROWC	total tramway infrastructure right-of-way in curves
TTROWS	total tramway infrastructure right-of-way in straight path
TTROWST	total tramway infrastructure right-of-way in stops' areas
t	travel time (run time)
t′	year of change of the corridor's operating frame
t_{fin}	year of the end of the economic life of a project
$t_{re} - t_{in}$	actual (recorded) temperature minus initial temperature of the rail
t_s	dwell time at stations/stops
t_{ts}	dwell time (waiting time) of trains at the two terminal stations of a route
U	track (normal) cant
U_{max}	maximum (normal) track cant
U_{th}	theoretical track cant
UAE	United Arab Emirates
UIC	Union International des Chemins de Fer (international union of railways)
UIC1, 2, 3, 4, 5, 6	track categorisation in accordance with UIC (based on the total daily traffic load)
USM	unsprung masses of the vehicle (one wheelset)
UTO	unattended train operation
UTS	ultimate tensile strength
V	train or vehicle or wheelset running or transit or forward speed
V_{amaxtr}	average permissible track speed
V_{ar}	average running speed
V_c	commercial speed
V_{CA}, V_{CB}, V_{CC}, V_{CD}, V_{CE}	commercial speed of tramways running on corridor categories A, B, C, D, E, respectively
V_{cmax}	maximum commercial speed
V_{cr}	vehicle critical speed
V_d	track design speed
V_{dmax}	maximum track design speed
V_{fr}	maximum speed for freight trains
V_{max}	train maximum running speed
V_{maxtr}	permissible track speed
V_{min}	running speed of the slowest trains circulating along a line – minimum running speed
V_{op}	train operating speed
V_p	train passage speed
V_{pas}	maximum speed of passenger trains
V_{pmax}	maximum train passage speed
V_{rs}	rolling stock design speed
V_t	train instant speed

V_1, V_2	relative velocities of the left and right wheels (in the direction of movement) of a railway wheelset
VAL	vibration acceleration level
VPF/VPC	value of preventing a fatality/casualty
VPI	value of preventing an injury
VVL	vibration velocity level
v_o	vibration velocity
v'_o	reference level of vibration velocity
W	total train resistance
W_{ac}	acceleration resistance
W_B	basic resistance
W_i	track gradient resistance
W_m	movement resistance
W_{Rc}	track curve resistance (drag)
W_{tr}	total track resistance
W_α	air resistance or aerodynamic resistance or aerodynamic drag
WILD	wheel impact load detector
X	longitudinal creep force applied on the wheel
X_{ij}	longitudinal creep forces exerted to the four wheels of a 2-axle bogie (i = 1,2 front and rear wheelset, respectively; j = 1,2 left and right wheels, respectively, in the direction of movement)
X_1, X_2	longitudinal creep forces exerted to left and right wheels (in the direction of movement) of a railway wheelset
x	longitudinal displacement of the wheelset
x′	derivative of the longitudinal displacement x of a railway wheelset
Y_1	total transversal force exerted on the rail via the wheel flange of the derailing wheel
y	lateral displacements of the wheelset in relation to its central equilibrium position
y_i	lateral displacements of the two wheelsets of a bogie (i = 1,2 front and rear wheelset, respectively)
y_o	lateral displacement of the wheelset in case of its radial positioning in curves (wheelset lateral offset)
y_w	oscillation wave amplitude (hunting of railway wheelset)
y′	derivative of the lateral displacement y of a railway wheelset
y''_{max}	maximum lateral acceleration of a railway wheelset
yy	derailment force axis
σ	flange way clearance
$\sigma(Q_{dyn1}, Q_{dyn2})$	typical deviation of the dynamic vertical forces of the sprung and semi-sprung masses of the vehicle
$\sigma(Q_{dyn3})$	typical deviation of the dynamic vertical forces of the un-sprung masses of the vehicle
ω	angular velocity of the two wheels of a conventional wheelset
ω_1, ω_2	angular velocities of the left and the right wheels (in the direction of movement) of a railway axle equipped with independently rotating wheels

γ_o	inclination of the tangent plane at the contact point between rail and wheel when the wheelset is in central position
γ_1, γ_2	angles formed by the horizontal plane, and the tangent planes at the contact points I_1 and I_2, respectively, when the railway wheelset is displaced from its central equilibrium position
γ_{nc}	lateral residual acceleration
γ_{ncmax}	maximum permitted lateral residual acceleration
γ_e	equivalent (or effective) conicity of the wheel
α	yaw angle of the wheelset
α'	derivative of the yaw angle α of a railway wheelset
α_{at}	angle of attack
α_{br}	coefficient of the vertical static loads of railway bridges
α_o	vibration acceleration
α'_o	reference level of vibration acceleration
α_s	sound-absorption coefficient
α_t	steel thermal expansion coefficient
$2\alpha_f$	angle of vertical displacement of the joint (sum of the angles that are formed by the two rails and the horizon)(rad)
ΣY	total transversal force
ΣQ	overall train weight
Π	adhesion force
π	constant equal to 3.14
μ	wheel–rail friction coefficient (adhesion coefficient, Coulomb coefficient)
Δ	distance between track centers (double track)
Δ_t	temperature change
$\Delta I_{max}/\Delta t$	maximum rate of change of cant deficiency
Δl	variation of the length of the rail
ΔP_{max}	maximum permissible change in pressure generated inside the tunnels
δ_p	angle of cant
φ	angle of rotation of the wheels and the axle
φ'	derivative of the angle of rotation φ of the wheels and the axle
φ_{bri}	dynamic coefficient for the loading of railway bridges (i = 2 or 3)
φ_o	road intersection angle
φ_t	tilting angle of car body
φ_1,φ_2	angles of rotation of two wheels of the same wheelset (axle with independently rotating wheels)
β	wheel–rail contact flange angle
β	coefficient that is empirically determined depending on the type of the superstructure wear
ν	exponent with values between 3 and 4
λ	coefficient

Chapter 1

The railway as a transport system

1.1 DEFINITION

The "railway" is a terrestrial mass transport system. Trains move on their own (diesel traction) or remotely transmitted power (electrical traction) using steel wheels* on a dedicated steel guideway defined by two parallel rails.

The railway transports passengers and freight. Its capability can extend to cover any distance in any environment (urban, suburban, periurban, regional and interurban). Its range for passengers' transportation is usually suited to approximately 1,500 km, while for freight the distances can be much greater.

From a transport system point of view, it is by default considered to comprise three constituents:

- Railway infrastructure
- Rolling stock
- Railway operation

1.2 CONSTITUENTS

1.2.1 Railway infrastructure

The term, railway infrastructure, describes the railway track and all the civil engineering structures and systems/premises that ensure the railway traffic (Figure 1.1).

The railway track consists of a series of components of varying stiffness that transfer the static and dynamic traffic loads to the foundation. Hence, the railway track comprises successively from top to bottom the rails, the sleepers, the ballast, the sub-ballast, the formation layer and the subgrade (Figures 1.2 and 1.3) (Giannakos, 2002; Profillidis, 2014).

The rails are mounted on the sleepers on top of elastic rail pads to which they are attached by means of a rail hold-down assembly called the rail fastening (Figure 1.4).

Rails, sleepers, fastenings, elastic pads, ballast and sub-ballast constitute the 'track superstructure', while the subgrade and the formation layer constitute the 'track substructure' (Figure 1.2).

The upper section of the track superstructure that comprises the rails, the sleepers, the fastenings and the rail pads forms what could be commonly called the 'track panel'. Switches and crossings by means of which the convergence, cross section, separation and joining of

* For a small number of metro lines and for many cases of driverless railway systems (cable-propelled and self-propelled) of low/medium transport capacity, rubber-tyred wheels are also used.

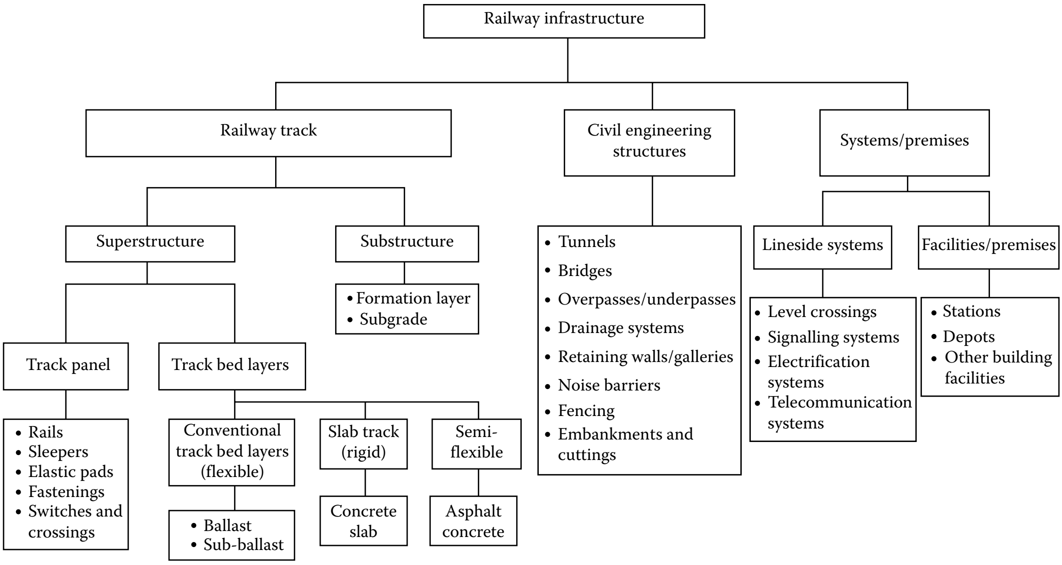


Figure 1.1 Constituents and components of the railway infrastructure.

tracks at specific points of the network is accomplished are also considered to be part of it (Figure 1.5).

The lower part of the track superstructure that comprises the ballast and its sublayers is called 'trackbed layers'. The trackbed layers and the track subgrade, considered as a whole, are called 'trackbed'.

Apart from the ballasted trackbed (conventional or flexible trackbed), a concrete trackbed (slab track or rigid trackbed) is more and more frequently used. The latter solution has proven to be very efficient in the case of underground track sections where maintenance requirements are greatly restricted (Figure 1.6).

A third trackbed system seldom applied is the 'asphalt concrete trackbed', or otherwise called 'semiflexible trackbed'. This solution is used in certain occasions in Italy and Japan for the construction of new high-speed lines. It is also extensively used in North America for the restoration of short lengths in critical segments of the track (tunnels, switches and

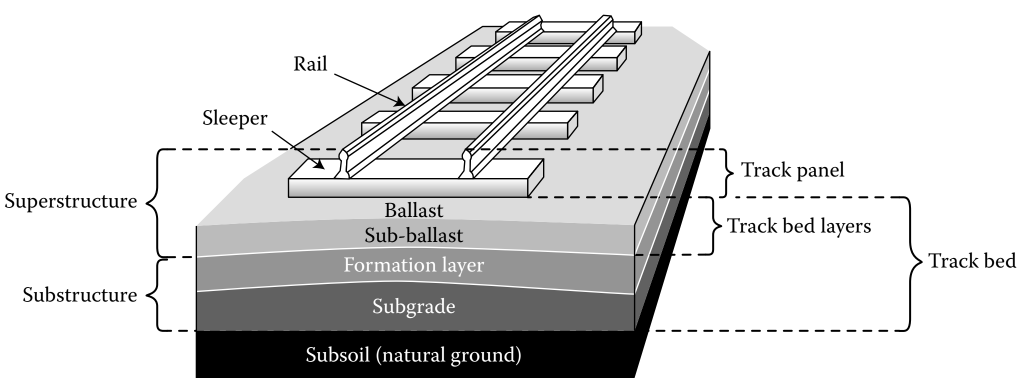


Figure 1.2 Railway track. (Adapted from Giannakos, K. 2002, *Actions in Railways*, Papazisi (in Greek), Athens.)

Figure 1.3 Railway track; ballasted track superstructure, Athens-E. Venizelos Airport suburban line, Greece. (Photo: A. Klonos.)

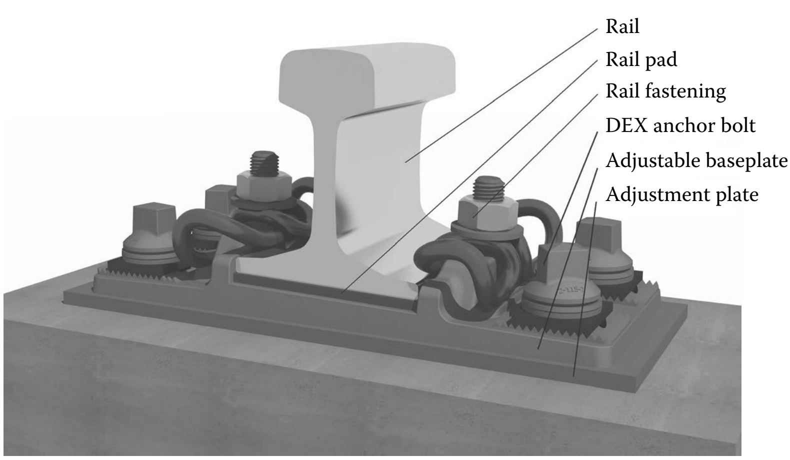


Figure 1.4 edilon)(sedra DFS (Direct Fastening System) with adjustable steel baseplate. (From edilon)(sedra, 2015.)

Figure 1.5 Switches and crossings configurations, Zurich, Switzerland. (Photo: A. Klonos.)

Figure 1.6 Slab track – Tunnel at Tempi, Greece. (Photo: A. Klonos.)

crossings sections, transition zones before or after major civil engineering works). Finally, this system is an alternative method for the improvement of the mechanical strength of the existing infrastructure (Buananno and Mele, 1996; Schoch, 2001).

The civil engineering structures comprise the tunnels and the underground sections of the track, the bridges, the overpasses/underpasses, the embankments and cuttings, the drainage systems, the soil retaining walls, the galleries, the noise barriers and the fences.

The track systems and premises are separated into

- Lineside systems that comprise the level crossings and the electrification, signalling and telecommunication systems.
- Facilities and premises that comprise the stations, the depots and other building facilities (administration buildings, warehouses, etc.).

Two special terms are usually used to describe the track structural characteristics along its length

'Plain' track (line): a segment of a railway track that does not have any junctions, crossovers, or points on it (http://en.wiktionary.org/wiki/plain_line, 2014).

'Open' track: a segment of a railway track that does not have any tunnels, bridges, overpasses, high embankments, deep cuttings and stations/stops on it.

1.2.2 Rolling stock

Rolling stock is the term employed to describe all railway vehicles, both powered and hauled, used either as power, trailer or engineering vehicles (Figure 1.7).

The power vehicles are self-propelled, that is, they are equipped with traction motors. These vehicles may

- Serve the sole purpose of hauling the trailer vehicles, and are then called 'locomotives' (or traction units).

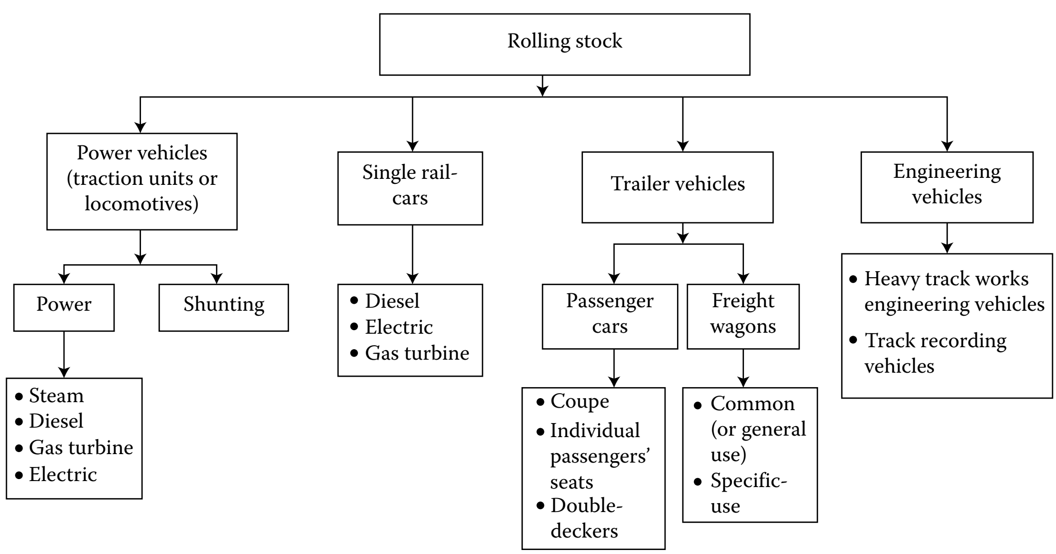


Figure 1.7 Categories of rolling stock.

- Transport a number of passengers, and are then called either ‘single railcars’ (they have a driver’s cab at one or both ends) or motor cars.
- To be used for shunting, hence they are called ‘shunting locomotives’.

Depending on their traction power used, locomotives are classified into four categories: steam locomotives, diesel locomotives, gas turbine and electric locomotives. Railcars are separated into electric, diesel and gas turbine railcars.

The trailer vehicles are not self-propelled. They serve the purpose of transporting people or goods. They may be classified into three basic categories depending on their function (Metzler, 1985):

- Passenger vehicles (or cars or coaches) intended to transport passengers.
- Freight vehicles (or wagons) intended to transport goods (common or general use freight wagons).
- Specific-use freight wagons intended for the transportation of certain types of freight only.

Also included among rolling stock are the engineering vehicles used to carry out track panel installation works and various track inspection and maintenance works. They are divided into two main categories:

- Heavy track works, engineering vehicles
- Track recording vehicles

Every railway vehicle, either trailer or power, consists of three basic parts (Figure 1.8):

- The car body (body shell)
- The bogies (trucks)
- The wheelsets (axle + 2 wheels)

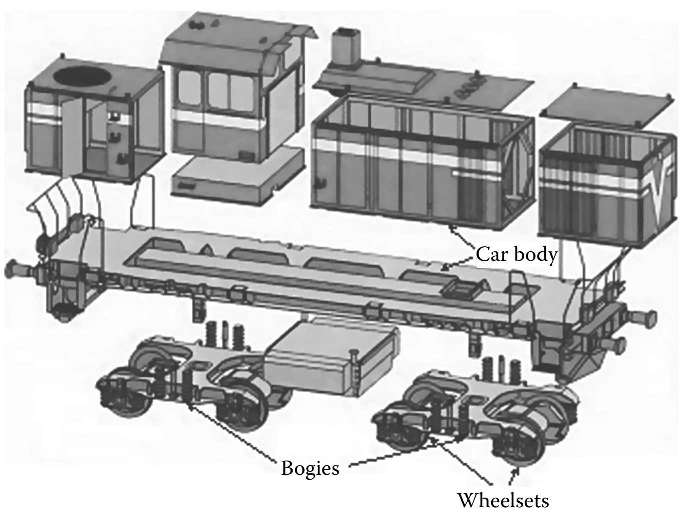


Figure 1.8 Main parts of a railway vehicle (locomotive): car body–bogies–wheelsets. (From Siemens, 2015.)

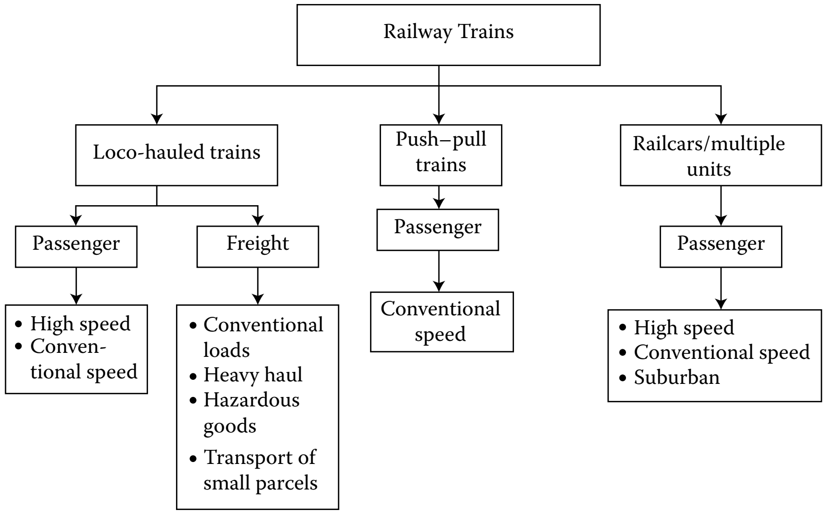


Figure 1.9 Types of railway trains.

The combination of locomotives and trailer vehicles forms the loco-hauled passenger or freight trains depending on the category of the trailer vehicles (Figure 1.9).

When two traction units are included in the same train formation then the operation is called '2-loco operation'.

The combination of single railcars, motor cars and/or trailer vehicles forms the railcars.

The railcars can move in both directions without the need of a shunting locomotive in contrast with loco-hauled trains which need a shunting locomotive.

Multiple units (MU) are diesel (DMU) or electric (EMU) trains fulfilling the following characteristics (Connor, 2014):

- Units are made of single railcars, motor cars and/or trailer vehicles semi-permanently coupled
- Driving cab is provided at each end of the unit. Drivers just change ends at the terminus
- Train length can be varied by adding or subtracting units
- Power equipment is distributed along the whole train (only motor cars and single railcars have power equipment)

Example formations of multiple units are

PR + TC + PR + PR + TC + PR, PR + MC + MC + PR, PR + PR + PR + PR

where

PR: Single railcar
TC: Trailer vehicle (car)
MC: Motor car

Push–pull trains are hauled passenger trains with (Figure 1.10):

- A locomotive at the front (pull–push) or at the rear (push–pull)

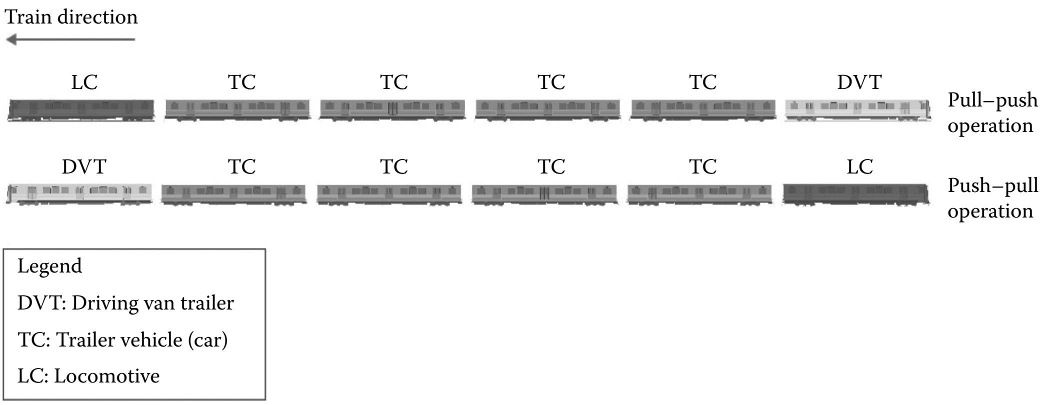


Figure 1.10 Push–pull train operation.

- A trailer vehicle at the rear or at the front, with a driving cab allowing the train to be driven from either end (Driving Van Trailer)
- A number of intermediate passenger trailer vehicles

Push–pull trains can move in both directions without the need of a shunting locomotive unit. The locomotive is controlled remotely through a train cable length when the Driving Van Trailer is leading (Connor, 2014).

1.2.3 Railway operation

The term railway operation describes all activities through which a railway company secures revenue service.

Railway operation may be broken down into technical and commercial. Figure 1.11 presents the activities of both technical and commercial operation.

Sound maintenance is a prerequisite for the smooth operation of the railway system. Maintenance is characterised as a 'horizontal activity', since it applies to all three constituents of the railway system.

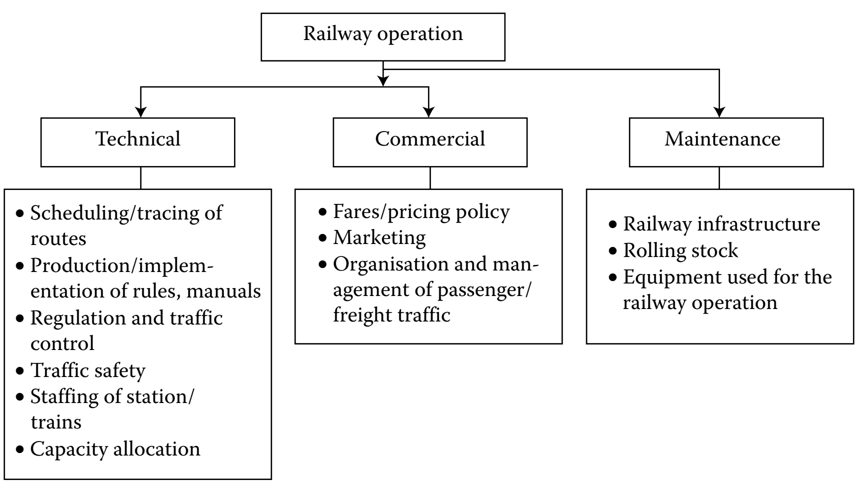


Figure 1.11 Technical and commercial railway operation activities.

1.3 THE RAILWAY SYSTEM TECHNIQUE

1.3.1 Description of the system

The two basic 'technical units' securing the 'transport' with railway means are the vehicle's wheelset and the rails (Figures 1.12 and 1.13).

The wheelset consists of three basic parts:

- The axle
- The wheels
- The axle boxes

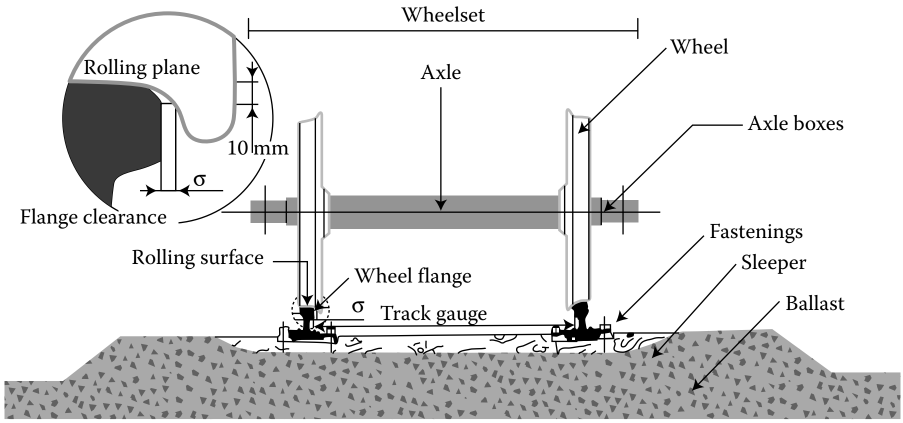


Figure 1.12 Rolling of a conventional railway wheelset on rails. (Adapted from Petit, J. M. 1989, *Conception des Bogies Modernes*, Revue ALSTHOM, Code APE 2811, France.)

Figure 1.13 Railway wheelsets, OSE, Piraeus factory, Greece. (Photo: A. Klonos.)

The wheels consist of

- The wheel tread, being the outer section of the wheels that allows the rolling on the rails
- The wheel body

Over the last 30 years, all freight and passenger vehicles running at speeds V > 160 km/h have been equipped with cast wheels (wheel rim and body being a monobloc).

The wheel flanges (one on each wheel) are characteristic of the inner section of the wheels; their mission is to prevent derailment in case the wheelset lateral displacement exceeds the limits set by the track gauge (Figures 1.12 and 1.14). Meanwhile, the flanges support the wheelsets' self-guidance when passing through switches and crossings configurations (Figure 1.15).

The wheel cross section (profile) is not orthogonal as it is in the case of road vehicle tyres. It features a slight variable conicity that 'is open' toward the inner part of the track (Figures 1.16 and 1.17).

The two wheels are rigidly connected through a cylindrical rod (axle) resulting in the rotation of both the wheels and the axle at the same angular velocity ω.

The above 'wheel–axle' system is called conventional (or classic); the wheelset runs a steel guideway consisting of two parallel rails set at a fixed distance between them (rail inside face) commonly called the track gauge (Figure 1.12).

The rail consists of three main parts (Figure 1.18):

- The head
- The web
- The foot

The upper surface of the rail head is curved, and hence, forms the surface over which the wheel treads run.

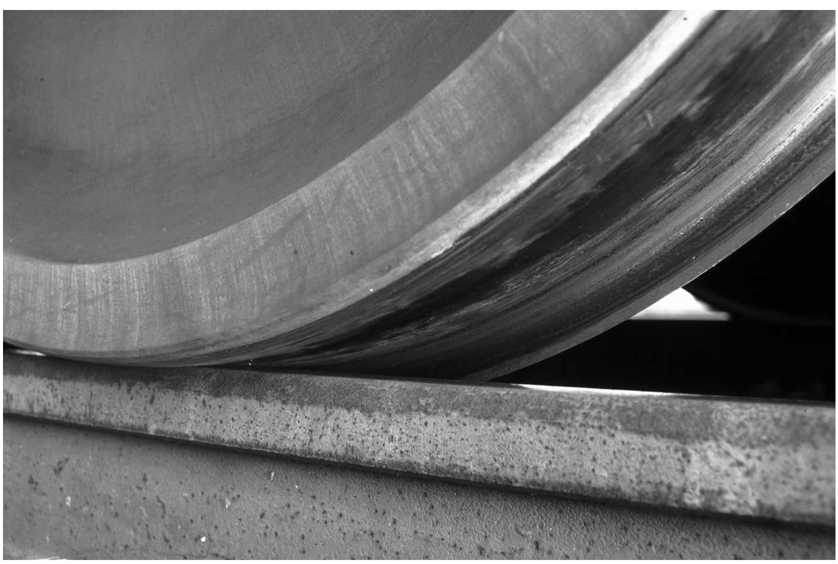


Figure 1.14 Railway wheel rolling on plain track. (Adapted from Vignal, B. 1982, SNCF Médiathèque, France.)

Figure 1.15 Railway wheel rolling on crossing's frog area. (Photo: A. Panagiotopoulos.)

Rails are mounted on sleepers at a certain angle, called rail inclination angle (usually 1:20 or 1:40) (Figure 1.19). This layout improves the transversal stability of the vehicles in a straight path.

1.3.2 Fundamental functional principles

During wheel rolling, elastic forces develop on the contact surface (creep, gravitational forces). Under smooth running conditions (good track ride quality, allowable speed limits, rolling stock in good condition), these forces guarantee the stability and guidance of vehicles on straight paths and in curves (see Chapter 2).

The generation of these forces comes about through

- The specific profile of the wheels
- The rigid connection of the wheels to the axle

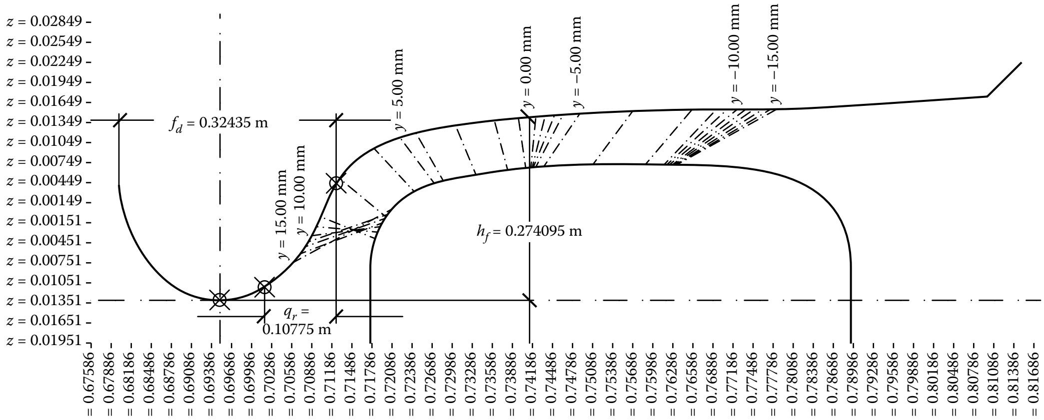


Figure 1.16 Wheel–rail contact surface geometry.

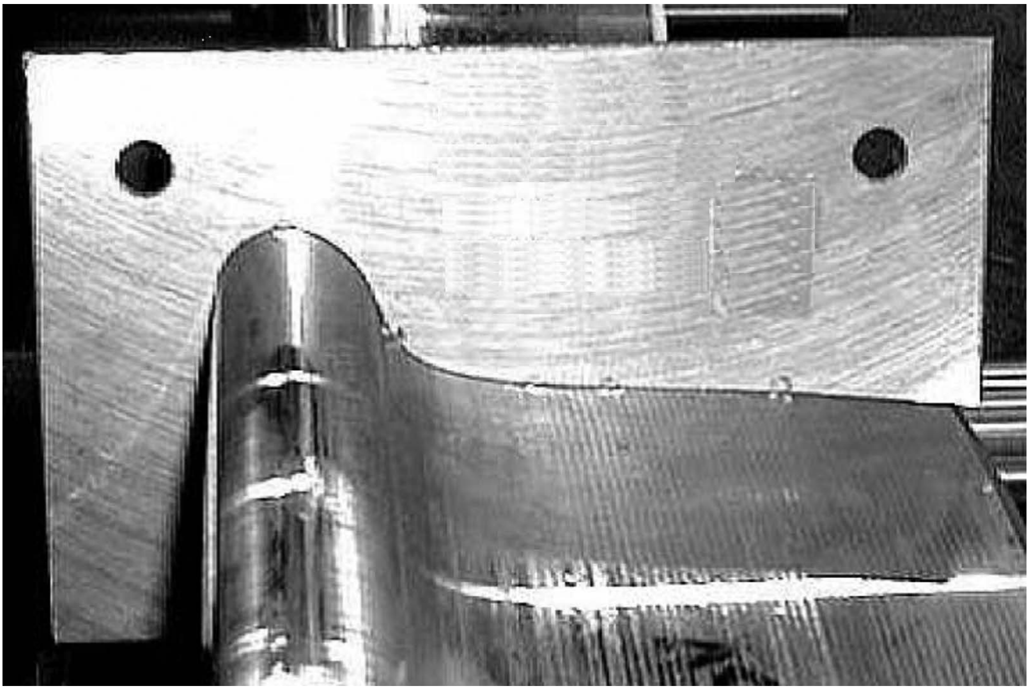


Figure 1.17 Geometry of rolling surface of the wheel.

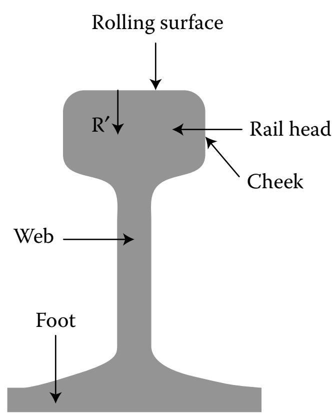


Figure 1.18 Main rail parts.

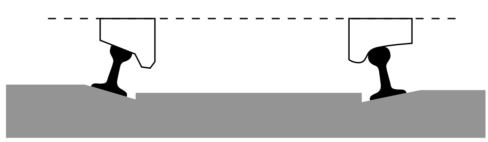


Figure 1.19 Inclined rails on sleepers.

- The geometry of the upper outer part of the rail head surface
- Creep phenomena

1.3.2.1 Running on a straight path

We consider a conventional railway wheelset centred on the track, running at a constant speed, V, on a straight path. If for any reason (track geometry defects, wheel asymmetry etc.), the axle is displaced transversally with regard to the initial equilibrium position, then, due to their profile, the two wheels roll with different radii ($r_1 \neq r_2$).

In the case of Figure 1.20 $r_2 > r_o > r_1$ applies (where r_o the rolling radius of the two wheels in the initial equilibrium position). Owing to the rigid connection of the wheels to the axle, both wheels have the same angular velocity, ω, and consequently the Equation 1.1 applies

$$\omega r_2 > \omega r_o > \omega r_1 \Rightarrow V_2 > V > V_1 \tag{1.1}$$

(V_1, V_2: relative velocities of the two wheels).

The wheel running at a relatively higher speed (wheel 2) will overtake the other wheel (wheel 1), and due to the rigid wheel connection, this will cause a rotation of the axle with regard to the transversal axis $\vec{y}_0$.

Owing to the simultaneous forward rolling of the wheelset, the rolling radius r_1 of wheel 1 constantly increases while the rolling radius r_2 of wheel 2 constantly decreases. When $r_1 > r_2$ wheel 1 starts overtaking wheel 2 the phenomenon is repeated.

This motion of the wheelset is known as 'hunting' oscillation.

In reality, the motion of a railway wheelset and, especially, of a complete vehicle (car body + bogies) is more complex. Owing to the simultaneous motion of the wheelset at a constant speed V, when the rolling direction of the wheels does not coincide with the vehicle displacement direction, friction forces are created on the wheel–rail contact surface (creep forces), and alter the kinematic behaviour described above, thereby giving the wheelset a dynamic behaviour (Esveld, 2001).

At very low speeds, this physical mechanism guarantees the stability of the vehicle (Pyrgidis, 1990; Moreau, 1992). Yet, at high speeds, high-amplitude oscillations are created and the motion becomes unstable. In such case, vehicle stability is secured, thanks to the longitudinal elastic connection between the bogies and the wheelsets (primary suspension), which limits the amplitude of oscillations. Should the wheel transversal displacements exceed the anticipated flangeway clearance, the rolling of the wheels on the rails is secured by the presence of wheel flanges (Figure 1.12).

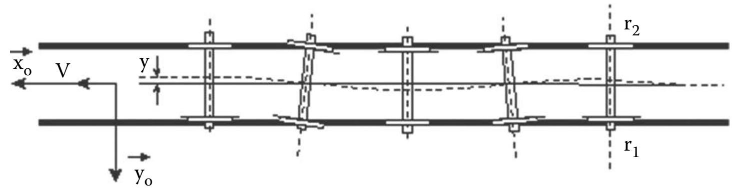


Figure 1.20 Sinusoidal motion of a railway wheelset (hunting). (Adapted from Petit, J. M. 1989, *Conception des Bogies Modernes*, Revue ALSTHOM, Code APE 2811, France.)

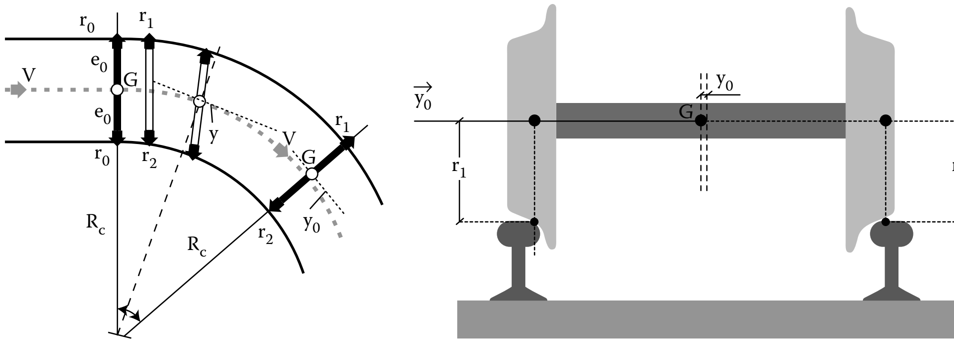


Figure 1.21 Movement of single railway wheelset in a curvature of track.

1.3.2.2 Running in curves

Let us examine the layout of Figure 1.21. Upon entering the curve, the wheelset is displaced by 'y' with regard to its outer face. Owing to the conic profile of the wheels, the initial rolling radius r_o of the two wheels changes into r_1 and r_2 for the outer wheel and the inner wheel, respectively.

The rolling radius of the outer wheel is bigger. The inequity $r_1 > r_2$ applies, and by extension, $V_1 > V_2$ also applies (where V_1, V_2: relative velocities of the two wheels).

Owing to the rigid connection of the wheels, the wheelset tends to rotate by itself toward the inner face of the curvature, displaced by y_o, seeking for a radial positioning inside the curve (the two wheels cover unequal paths).

As in the case of running on straight sections of track, when the wheelset transversal offset exceeds the existing flangeway clearance 'σ', the running of the wheels on the rails is secured by the presence of flanges.

The motion described above concerns a single isolated wheelset. The bogie negotiation in curves is more complex and the axle's positioning is affected by the motions of both the bogies and the car body. However, the wheelset inscription mechanism in curves remains the same.

Since the birth of the railway (1825) till now, the system described above represents the basic unit materialising the physical mechanism of guidance of the railway vehicles on a straight path and in curves. Unlike other means of transport, railway vehicles do not require human intervention (steering wheel operation) or complicated mechanisms.

At this point the following classification of curves is proposed, based on the range of the horizontal radii values R_c.

- $R_c \geq 5{,}000$ m — Very big radii
- $2{,}000 \text{ m} \leq R_c < 5{,}000$ m — Big radii
- $500 \text{ m} \leq R_c < 2{,}000$ m — Medium radii
- $250 \text{ m} \leq R_c < 500$ m — Small radii
- $100 \text{ m} \leq R_c < 250$ m — Very small radii
- $20 \text{ m} \leq R_c < 100$ m — Tramway network radii

1.3.3 Distinctive features of railway systems compared to road means of transport

As a system of mass terrestrial transportation, the railway differs from the road means vis-à-vis its three constituents, that is, the railway infrastructure, the rolling stock and railway operation.

Indicatively:

- The railway has only one degree of freedom. The one degree of freedom facilitates the automation of a range of operations such as driving, signalling, braking, electrification. Conversely, unlike the road vehicle, the railway cannot provide 'door to door' services (rigid system).
- Owing to the low adherence between wheel and rail (steel/steel contact) and the greater braking weight, the braking distance of a train is, for the same speed, much greater than that of a road vehicle; since braking seldom prevents a collision, it is of great importance that the railway can 'prevent' such accidents by taking those measures necessary in order to avoid collision condition.
- On the road arteries, the traffic lights are virtually always time regulated (time period for traffic signals). The opposite applies to the railway where regulating of the trains is based on the location of the railway vehicles.
- Railway vehicles, by contrast with road vehicles, do not need to be guided by human intervention (steering). The direction of movement is determined by the steel guideway only.
- Trains possess operational and constructional features which increase the aerodynamic phenomena as they move (high speed, great length, large frontal cross section). These phenomena may have negative consequences on the rolling stock, the passengers, the users of the system who are on the platform and the staff working near the track.

1.4 CLASSIFICATION OF RAILWAY SYSTEMS

Railway systems can be classified in to many ways. This section defines the term 'speed' in railway engineering and attempts a classification of railway systems based on their functionality, the track gauge and the traffic. This classification is consistent with the structure of this book and is aimed at facilitating the reader.

1.4.1 Speed in railway engineering: Design and operational considerations

The term 'speed' in a railway context may be defined in various ways, depending on the technical and/or operational context being considered. The following definitions are commonly used:

- *Track design speed* (V_d), which is defined as the speed the track alignment and corresponding railway infrastructure as a whole (superstructure, substructure, civil engineering structures, systems/premises) has been designed and constructed for. Thus, it is regarded as the maximum speed a train can safely and comfortably operate at on a given track. This speed is not related to any operational or track capacity constraints.
- *Permissible track speed* (V_{maxtr}), which is defined as the maximum speed that may be developed on a railway track section at the time a given rolling stock is commissioned. This speed is determined by the Infrastructure Manager of a railway network taking in consideration the track ride quality as well as other performance aspects at the moment. The permissible track speed is directly related to the maintenance level of the track and the line as a whole.

- *Maximum running speed* (V_{max}), which is defined as the maximum speed developed by a particular train type on a given line, while performing a scheduled route. This speed may either refer to a small segment of the line, or it may develop at the biggest part of the route.
- *Operating speed* (V_{op}), which is defined as the speed that is developed at the biggest part of the route (e.g., at 2/3 of the route length) by a particular train type while performing a scheduled route. *Passage speed* (V_p), which is defined as the constant speed with which a train passes from a particular, characteristic segment of the line which is of small length (e.g., passing through a tunnel, passing through stations, etc.).
- *Instant speed* (V_t), which is defined as the speed with which a train passes from a specific kilometric point at a specific time.
- *Commercial speed* (V_c), which is defined as the ratio of the length of a railway route (usually between the two terminals or between two important intermediate stations) to the time it takes to cover it, including halt times at all intermediate stations and delays. Commercial speed always refers to a particular type of train and a given route.
- *Average running speed* (V_{ar}), which is defined as the quotient of the length of a line segment (usually between two successive stations), to the time taken to pass this segment, considering normal traffic conditions (e.g., no unforeseen delays). The average running speed always refers to a particular train type and a given line segment.
- *Rolling stock design speed* (V_{rs}), which is defined as the maximum speed that, according to the manufacturer, can be developed by a particular type of locomotive, or with which a trailer vehicle can move, or, finally, the maximum speed that can be developed by a multiple unit of given formation taking into consideration the traction system (diesel or electric power), the hauled weight, the track geometry alignment design and considering the track to be of very good ride quality.

It should be noted that it is desirable that the track design speed (V_d) be the same on all track sections of a railway corridor.

The mathematical expressions (1.2) generally apply:

$$V_{ar} \le V_{maxtr} \le V_d \tag{1.2}$$

Regarding speed, the quality of the railway infrastructure is secured when

- V_{maxtr} at individual track segments coincides with the track design speed V_d which, however, corresponds to a particular traction system
- The average running speed V_{ar} is nearly equal to V_{maxtr} (in case this refers to a segment between two successive stations due to the train's acceleration and deceleration while starting and stopping, these two speed values cannot coincide, and in this occasion the value of V_{maxtr} is greater)

In regard to the combination of track and rolling stock, the design speed of the rolling stock (V_{rs}) must be slightly greater than V_d or at least equal to V_d.

Finally, in regard to the level of service and, more specifically, the run times, the maximum train running speed V_{max} must be achieved for the longest part of the route.

Figure 1.22 illustrates the graphical representation of the speeds V_d, V_{maxtr}, V_{ar}, V_c, for a route AB with an intermediate stop, considering that the speeds $V_{d,}$ V_{maxtr} are the same for all of the route length S.

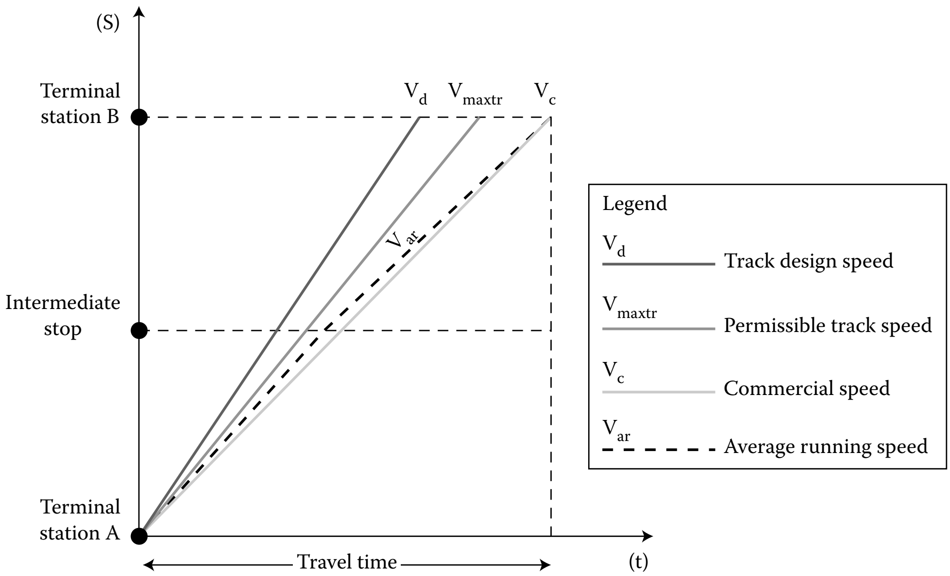


Figure 1.22 Speed in railway engineering.

1.4.2 Classification of railway systems based on functionality

In general, the railway systems fall under the category of terrestrial transport moving along a dedicated corridor ('fixed permanent way').

Depending on the rolling system they use, these guided transport modes are distinguished into railway, aerial, road and magnetic systems (Figure 1.23).

The term 'railway means of transport' may include all transport means whose rolling system involves at least one iron component (steel wheels on rails, or rubber-tyred wheels on a steel guideway). In this context, this book examines only those means of transport that have this particular characteristic in common.

The railway systems transport passengers (passenger railway systems) or goods (freight trains).

On the basis of the geographic/urban environment in which they operate, and generally on their functionality, they are separated into (Figures 1.24 through 1.37)

- Interurban systems
- Suburban/commuter (urban rail)/regional systems
- Urban systems
- Steep gradient railway systems

The *interurban railway* serves trips greater than 150 km and usually links major urban centres. It comprises high-speed trains ($V_{max} \geq 200$ km/h, $V_c \geq 150$ km/h) (Figure 1.24) (see Chapter 12) and conventional speed trains ($V_{max} < 200$ km/h) (Figure 1.25).

The suburban railway (Figures 1.26 and 1.27) is a railway means of transport usually running on electrified lines with characteristics adapted to commuter services within the impact limits of major urban areas (suburbs and satellite regional centres). Its range can exceed 100 km and may even reach up to 150 km. The nomenclature varies. When covering

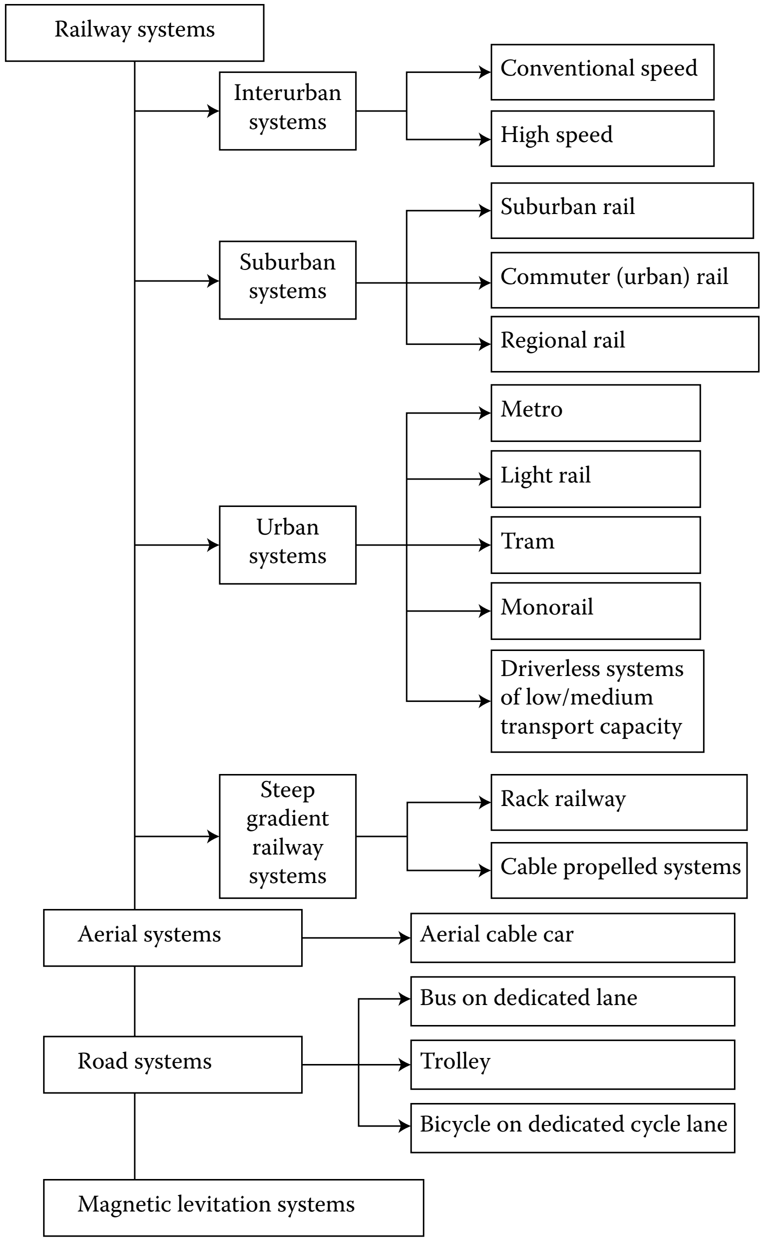


Figure 1.23 Classification of guided transport modes on 'fixed permanent way'.

distances of 30–50 km, it is designated as an urban railway, whereas when it covers longer distances it is called sometimes 'regional' railway.

Urban railway systems include

- The metro (Figures 1.28 and 1.29)
- The light metro (Figure 1.30)
- The tramway (Figure 1.31)

Figure 1.24 High-speed interurban railway (THALYS). (Photo: A. Klonos.)

Figure 1.25 Conventional speed interurban train, Helsinki, Finland (hauled electric passenger train). (Photo: A. Klonos.)

Figure 1.26 Suburban railway, Berlin, Germany (double-deck electric railcar). (Photo: A. Klonos.)

Figure 1.27 Suburban railway, Berlin, Germany (electric railcar). (Photo: A. Klonos.)

Figure 1.28 Metro (with driver), Athens, Greece. (Photo: A. Klonos.)

Figure 1.29 Metro (driverless), Paris, France. (Photo: A. Klonos.)

Figure 1.30 Driverless light metro, Copenhagen, Denmark. (Photo: A. Panagiotopoulos.)

Figure 1.31 Modern tramway, Athens, Greece. (Photo: A. Klonos.)

Figure 1.32 Monorail (suspended system), Memphis, Tennessee, USA. (Adapted from Nightryder84 on the English Wikipedia, 2005, available online at : https://commons.wikimedia.org/wiki/File:Memphis_front_view.jpg.)

Figure 1.33 Monorail (straddled system), Sao Paolo, Brazil. (Adapted from Hpeterswald, 2013, available online at: https://commons.wikimedia.org/wiki/File:Metro_Monorail_Pitt_Street.jpg (accessed 8th August 2015).)

- The monorail systems (Figures 1.32 and 1.33)
- The driverless railway systems of low/medium transport capacity (Figures 1.34 and 1.35)

Out of the above systems, the first three serve trips which are performed exclusively within a city (urban transport), whereas the two latter ones are mainly used for trips with a different character. More specifically

- In essence, metros move underground and are characterised by great transport capacity and high implementation cost.

Figure 1.34 Driverless self-propelled railway systems of low/medium transport capacity. (From POSCO ICT Co., Ltd., 2015.)

Figure 1.35 Driverless cable-propelled railway systems of medium transport capacity – Birmingham airport system, UK. (Adapted from Doppelmayr Cable Car, 2008, available online at: https://en.wikipedia.org/wiki/Cable Liner (accessed 7th August 2015).)

- Trams are integrated in the road arteries of the city, using a specific track superstructure.
- The light metro is, on the basis of its construction and operation features, a system somewhere between the tram and the metro. Light metro and tram belong to the so-called ‘Light Rail Transport Systems’ (Light Rail Vehicles – LRV or Light Rail Transport Systems – LRTs).

The monorail moves using a system of rubber tyres (this is the most common type) on an elevated guideway comprising a single beam made of concrete or steel. It serves short distances within the urban environment and is particularly suitable for trips within recreation areas (thematic parks, zoo parks, etc.), as well as for connecting the city centre to the airport.

Finally, driverless systems of low/medium transport capacity move on an exclusive transport corridor using either single vehicles with a transport capacity of 3–25 persons (Figure 1.34), or trains of low and medium transport capacity. They are either cable propelled (Figure 1.35) or self-propelled electric systems and they belong in the category of Automated People Movers. In the urban environment, such systems may serve as feeders for heavy rail transport systems. However, they usually operate for the service of trips within airports, large hotels, casinos, congress centres and health centres, educational institutions and big companies’ premises.

The ‘steep gradient railway’ serves small-distance connections with an important difference of altitude between the two edges of the railway line. They are separated into rack railways (Figure 1.36) and cable-propelled railway systems (Figure 1.37).

Figure 1.36 Cog railway, Arth Goldau, Switzerland. (Photo: A. Klonos.)

Figure 1.37 Funicular, Graz, Austria. (Adapted from Riemer, J.F. 2007, available online at: http://en.wikipedia.org/wiki/Schlossbergbahn_(Graz) (accessed 7th August 2015).)

The rack (or cog) railway is mainly used to approach remote mountain developments and tourist resorts, on tracks with longitudinal slopes usually exceeding 50–70‰. Apart from the two classical rails, the cog railway superstructure includes a special toothed rack rail mounted between the conventional running rails. The wheelsets of the power vehicles are fitted with one or more cog wheels that bind in the rack rail. The required supplementary traction force is achieved through the engagement of the rack rail teeth with the locomotive pinion teeth.

Cable-propelled railway systems for steep gradients use vehicles that are hauled via cables. On the basis of the technique that is used for their traction they are divided into funicular (nondetachable, cable-propelled vehicles for steep gradients), cable railway (detachable, cable-propelled vehicles for steep gradients), and inclined elevator.

The funicular (Figure 1.37) operates using two vehicles which move on rails with the aid of a cable; one of the vehicles is ascending while the other one is descending. The cable rolls over pulleys which are mounted on the track superstructure. The vehicles are permanently connected to both ends of the cable and start and stop simultaneously. The ascending vehicle uses the gravitational force of the descending one (counterbalance system). The system usually connects distances of less than 5 km, with constant longitudinal gradients of around 300–500‰ (max value recorded in practice: 1,200‰, see Chapter 10).

The cable railway also uses vehicles which run on conventional rails, using a cable which moves constantly and at a constant speed. The difference between the two systems lies in the fact that for the cable railway the vehicles are not permanently connected to the cable. The vehicles can stop independently, disconnecting from the cable, and may start again, reconnecting to the cable. This process may occur either automatically or manually (San Francisco system, USA).

The inclined elevator or inclined lift or inclinator is a variant of the funicular. It operates using a single vehicle which is either winched up at the station on the top of the inclined section where the cable is wound on a winch drum, or the weight of the single vehicle is balanced by a counterweight so that the system operates as a funicular. It usually connects distances of less than 1.5 km, with constant longitudinal gradients of around 500–700‰. This system can serve extremely steep gradients.

Freight trains, as seen in Figure 1.9, are divided into the following categories:

- Conventional loads (axle load $Q \leq 25$)
- Heavy loads (axle load $Q > 25$)
- Hazardous goods
- Transport of small parcels

1.4.3 Classification of railway systems based on track gauge

The track gauge (2e) is the distance between the inner edges of the heads of the two rails measured at 14–16 mm below the rolling surface plane (Figure 1.38).

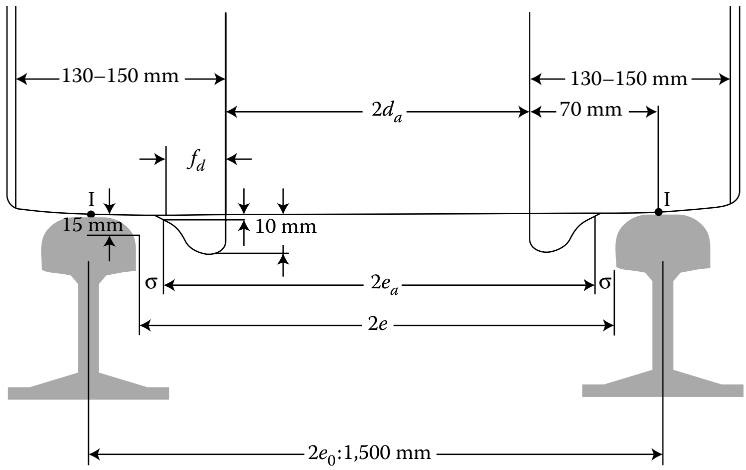
The track gauge is not the same in all countries. In many countries it varies from region to region.

On the basis of the gauge, railway lines are divided in to five categories (Esveld, 2001):

- *Standard tracks or standard track gauge*: This category mainly comprises the 1435 mm gauge. This distance (4 feet and 8 inches) was established by the British engineer George Stephenson (1781–1848).
- *Broad tracks or broad track gauge*: This category mainly comprises the following gauges: 1,520/1,524 mm (former Soviet countries), 1,600 mm (Irish gauge) and 1,665 mm, 1,667 mm.
- *Meter tracks or meter track gauge*: This category mainly comprises the following gauges: 914 mm, 950 mm, 1,000 mm (meter), 1,050 mm, 1,067 mm (Cape gauge).
- *Narrow tracks or narrow track gauge*: This category comprises gauges from 600 to 900 mm and mainly 600 mm (Decauville), 700 mm, 750 mm, 760 mm (Bosnian gauge). These gauges are usually used for secondary lines (industrial area, factory, mine service lines). Meter and narrow tracks are also termed as small gauge tracks.
- *Mixed gauge tracks*: This category comprises tracks on which trains of different gauge category may run simultaneously.

Regardless from the track gauge category, the distance between the rails remains constant throughout the network length apart from the curved alignment sections with small curvature radii ($R_c < 150$–200 m), where in many cases, a widening of the track gauge is permitted to facilitate the inscription of vehicle wheelsets (gauge widening) (Pyrgidis, 2005).

Figure 1.38 illustrates the layout of the rails and the wheels that roll on them, for the case of standard track gauge (indicatively).



2e:	Track gauge	=1,432–1,470 (mm) +0
$2e_a$:	Outer flange edge-to-edge distance (flange gauge)	=1,426 (mm) −16 +3
$2d_a$:	Back-to-back wheel distance (inside gauge)	=1,360 (mm) −3
$2e_o$:	Theoretical distance between the running surfaces of the right and the left wheel when centred	≈1,500 (mm)
f_d:	Flange thickness	= 33–25 (mm) (wear limit = 25 mm)
σ:	Flange way clearance	$= (2e - 2e_a)/2$ (mm)
2σ:	Total flange way clearance	$= 2e - 2e_a$ (mm)

Figure 1.38 Railway wheels on rails–track of standard gauge–geometrical and constructional dimensions (track-centred wheelset). (Adapted from Alias, J. 1977 *La voie ferrée*, Eyrolles, Paris.)

The mathematical equation (1.3) applies

$$2\sigma = 2e - (2d_a + 2f_d) \tag{1.3}$$

1.4.4 Classification of railway systems based on traffic

The number and percentage of each category of rolling stock (railway traffic composition) using a specific railway infrastructure can be directly used in classifying railway systems.

On the basis of their traffic, railway network/corridors can be classified in five categories as follows (Christogiannis, 2012; Christogiannis and Pyrgidis, 2013):

1. Exclusively used by freight trains (freight-dedicated network/corridor)
2. Mainly used by freight trains
3. Network/corridor with mixed traffic operation
4. Mainly used by passenger trains
5. Exclusively used by passenger trains (passenger-dedicated network/corridor)

The exact limits that are used in order to distinguish networks/corridors based on their traffic composition are set in Chapter 17.

The term '*mixed traffic operation*' commonly describes the routing of both freight and passenger trains on the same track.

The term '*exclusively used or dedicated*' describes the exclusive routing of either passenger or freight trains on the track.

1.5 THE CAPABILITIES OF THE RAILWAY SYSTEM

1.5.1 Advantages and disadvantages of the railway

Table 1.1 presents the advantages and disadvantages of the railway compared with other means of transportation.

Some of the advantages/disadvantages of the railway system are discussed below, while Section 1.5.3 presents a comparison between the level of service provided by the railway and comparison with that of other competing transportation systems.

- *High transportation capacity*: The steel-on-steel contact significantly reduces the specific rolling resistance (15 N/t for the railway, 150 N/t for a tourist coach, 300 N/t for a road truck) (Metzler, 1981).
  A locomotive can therefore pull a greater load than a road vehicle by applying the same tractive effort. Also, a train is formed of many vehicles, thus allowing it to increase or alter its transporting capacity according to the demand.
  For example, in order to transport 700 passengers, a train of length between 280 and 300 m is needed. For the same number of passengers to be transported by road it would require
  - 15 coaches with 44 seats covering a length of road of 1,050 m.
  - 170 private cars with 4 seats in each, covering a road distance of 11,900 m including any safety distance.
- *High-speed travel*: Nowadays, technical advances in the areas of rolling stock and the track allow a train to move safely on a straight track of good rolling conditions at running speeds $V_{max} > 300$ km/h (since 2009, $V_{max} = 350$ km/h in China. However, following the fatal rail accident which occurred on the Ningbo–Wenzhou line on the 23 July 2011, the maximum running speed of high-speed trains was initially limited to $V_{max} = 300$ km/h. However, there is potential to allow an increase in the maximum running speed to its initial value).

Table 1.1 Advantages and disadvantages of the railway

Advantages	*Disadvantages*
• High transportation capacity • High-speed travel • Travel safety • Rail services regardless of weather conditions (regularity of services) • Environmentally friendly transport • One degree of freedom (automation of many operations) • Passenger comfort/relaxed state of mind • Small land take (right-of-way)	• Increased requirements in track geometry design (horizontal, longitudinal alignment) • Low wheel–rail adhesion coefficient • One degree of freedom (no door-to-door services) • Hard (noisy) rolling • Low network density

Table 1.2 Railways – characteristic speeds

Characteristic speeds	*Maximum value (km/h)*
Rolling stock design speed (V_{rs})	380 (CRH380D & DL ZEFIRO)
Average running speed between successive stops (V_{ar})	283.4 (China)
Maximum running speed (V_{max})	320 (Europe, Japan), 300(350) (China)
Track design speed (V_d)	400
Speed record	574.8 (France)

China holds the record for the fastest average running speed (between two successive intermediate stops) at V_{ar} = 283.4 km/h, based on 2015 data (Hartill, 2015), while France retains the world speed record with a test train (574.8 km/h) recorded on April 3, 2007.

With regard to passenger transportation, trains in many countries move on conventional lines at running speeds in excess of 160 km/h, while there are 21 countries globally operating high-speed lines ($V_{max} \geq 200$ km/h, and $V_{ar} \geq 150$ km/h, see Chapter 12).

Finally, there are 11 countries worldwide that have lines operating with $V_{max} \geq 250$ km/h and $V_{ar} \geq 200$ km/h.

As far as freight transport is concerned, numerous countries operate trains moving between 100 and 120 km/h. Table 1.2 shows values for characteristic speeds which apply today for the railway globally.

- *Rail services regardless of weather conditions (Figure 1.39)*: Safety in train operation and train movement is generally not affected by extreme weather conditions (fog, snow, ice, strong winds), and cancellation of scheduled services due to weather conditions is seldom needed. Given this fact, the railway ensures regularity in its services, a quality of great importance to its users.

Figure 1.39 Rail services regardless of weather conditions. (Photo: A. Klonos.)

- *Passenger comfort/relaxed state of mind*: Provided that it offers a satisfactory level of service, the railway is generally viewed in a positive light in comparison with road and air transport, as the passenger
  - Has greater comfort in terms of space when the train is moving; he/she can move about more, visit the restaurant car, work on the train. It should be noted that for high-speed trains there is no obligation for passengers to wear a seatbelt (http://www.railwaygazette.com/news/single-view/view/study says no to seat belts on trains.html).
  - Can enjoy the view throughout the whole journey.
  - Is transported 'on terra firma' without having to drive him/herself, which psychologically is more comforting.
- *Occupies a small space*: A double standard gauge track occupies a space of about 1/3 of that of a two-way highway with three lanes per direction (Figures 1.40 and 1.41). Indicatively, for 1 km of high-speed railway line, 3.2 hectares of land are needed, while for the same amount of highway length 9.3 hectares of land are needed.
- *Increased requirements in track geometry alignment*: The design of a railway line is more demanding both in terms of the horizontal alignment and in terms of its longitudinal alignment compared with that of a road. Regarding its horizontal plane, the curve radii for the interurban and suburban railway must be greater than $R_c \geq 250$–300 m in 'open' track sections (outside the area of stations).

  In terms of its longitudinal alignment, the effective exploitation of a railway network sets the gradients for the interurban and suburban railway at $i_{max} = 3\%–4\%$ with usual gradients at less than 2%–2.5%. In the case of road-works, the corresponding values lie between 8% and 10%.

  Table 1.3 shows the characteristic gradient values for various means of transport and networks.
- *Low wheel–rail adhesion coefficient*: In railways, the contact surface between the rail and the wheel features a small adhesion coefficient due to the nature of the materials in

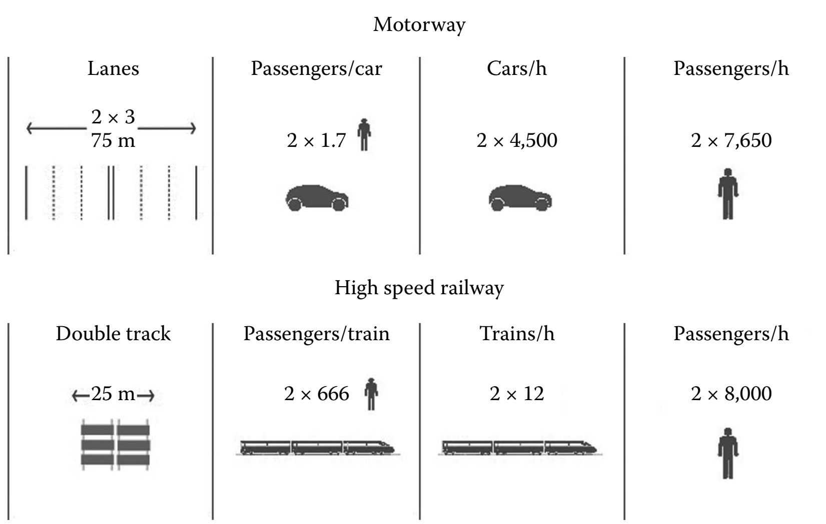


Figure 1.40 Comparison between occupied space (right-of-way) of a double standard gauge track and a highway for approximately the same transport capacity. (Adapted from UIC 2010, High speed rail-fast track to sustainable mobility, Paris, available online at: www.uic.org/download.php/…/521E.pdf (accessed 20th March 2015).)

Figure 1.41 Right-of-way for a double railway track and for a highway (2×3), Mundener bridge, Koln-Frankfurt, Germany. (Photo: A. Klonos.)

contact (steel on steel). In road transport, this coefficient is approximately three times greater (Metzler, 1981).

The small adhesion coefficient acts negatively on two basic operations: braking and starting the train. The greatest braking distance required to stop the train automatically sets a maximum speed limit as well as a maximum longitudinal gradient for the railway.

Furthermore, the wear on the wheel–rail contact surface created by the friction between the wheel and the brakes is a major financial burden for the maintenance and operation of a railway network.

Table 1.3 Characteristic values of longitudinal gradients for different means of transport and network cases

Means of transport/network	*Longitudinal gradient*
Road transport	8% (a road category, V = 60 km/h) 15%–20% (maximum values)
Cog railway	5%–48%
Cable-propelled systems (funicular)	9%–120%
Tramway	7%–8% (maximum values)
Metro	5% (maximum value)
Monorail	10% (maximum value, 20% gradient is used in two cases) (see Chapter 6)
High-speed network	3.5%–4.0% (maximum values)
Conventional interurban/suburban train	3%–4% (maximum values)

Table 1.4 Braking distance for different means of transport

Transport means	*Braking distance*
Boeing 747, landing speed V = 200 km/h	1,500 m
Road vehicle, V = 120 km/h	
Dry road surface	95 m
Wet road surface	142.5 m
Lorry, V = 80 km/h	
Dry road surface	60 m
Wet road surface	90 m
Freight train, V = 80 km/h	700 m (emergency brake)
TGV (PSE), V = 270 km/h	3,000 m (emergency brake)
TGV A, V = 300 km/h	3,200–3,500 m (emergency brake)

As a result of the lower adhesion coefficient the necessary braking length for the same speed and weight is greater for a train than it is for a private car.

Table 1.4 presents the braking distance for various categories of transport. Table 1.5 presents the proportional relations.

- *Hard (noisy) rolling*: The hard steel-on-steel rolling, which characterises the wheel–rail system, increases rolling noise and vibrations, resulting in the requirement for mitigation measures, regarding both the source of the noise (rolling stock – track) and the receptor.
- *Low network density*: It is possible at specific points of the railway network to converge, cross, split and join tracks. The above conditions are achieved through the various switches and crossings configurations provided as part of the superstructure along the section in question.

  However, it is both technically difficult (if not impossible) and economically unprofitable to develop a rail network that has the same level of density as a road network.

1.5.2 Comparison of the characteristics of railway systems

Table 1.6 presents the basic technical and operational characteristics of passenger railway transportation systems. These characteristics are analysed in depth in the respective book chapters devoted to each system.

1.5.3 Comparison of the capabilities of different transportation systems

The advantages and disadvantages of each transport system are usually compared by quantifying the parameters of one system and contrasting them with the corresponding ones of

Table 1.5 Comparative braking distances among different modes

	Aeroplane/ high-speed train	*Road vehicle/ conventional-speed train*	*Truck/freight train*
Braking distance ratios	1:2	1:10	1:10

Table 1.6 Main technical and operational characteristics of passenger railway transport systems

Technical and operational characteristics	*High-speed interurban rail*	*Conventional interurban rail*	*Suburban/ urban/ regional rail*	*Metro*	*Tramway*	*Monorail*	*Cog railway (pure rack system)*	*Funicular*
Route length (km)	>300	>150	10–30/30–50/50–150	10–30	5–20	1.5–12	4–20 (Max 19.09, usually 4.5–6)	Usually <1.2 (Max 4.827)
Track gauge	Normal Broad	All gauges	All gauges	Normal Metric	Normal Metric	Beam of 2.30–3.0 m width	All gauges (usually metric)	Various gauges (usually metric)
Number of tracks	Double	Double Single	Double (suburban/ urban rail)	Double	Double	Usually 2 beams at sufficient distance	Usually single	Single with passing loop. Double with or without passing loop
Traction system	Electric	Electric Diesel	Usually electric	Electric	Electric (overhead wires, through the ground, with energy storage systems)	Electric motors setting a system of rubber tyred wheels to roll	Usually electric Diesel Bio-diesel Steam	Via pulled electric cables placed on the track superstructure (Nondetachable vehicles – counterbalance system)
Distance between successive stops (m)	Large distance (normally 1 station every 200 km)	Normally 1 station every 75 km	Normally 2,000–3,000 (suburban)	500–1,000	400–600	800–1,500	Rarely, 1–2 intermediate stations	Without intermediate stops 1–2 stops located antisymmetrically for the two running vehicles
Commercial speed (km/h)	≥150	<150	>45–50/50–60/60–80	30–40	15–25	20–40	7.5–20	Usually <20 (5.5–50)
Longitudinal gradient (%)	0–4	0–3.5	0–3.5	0–5	0–8	0–10 (Max 20)	>5 (Maximum 48, usually 20–25)	> 10 (Max120, usually 30–50)
Frequency (Headway)	Depending on demand	Depending on demand	<60 min (suburban: 10–30 min Min headway: 90 sec)	<15 min 2–8 min Min headway: 60 sec	<20 min 5–15 min Min headway: 90 sec	<20 min 3–15 min Min headway: 60 sec	Depending on demand	Frequent services depending on length of trip

(*Continued*)

Table 1.6 (Continued) Main technical and operational characteristics of passenger railway transport systems

Technical and operational characteristics	*High-speed interurban rail*	*Conventional interurban rail*	*Suburban/ urban/regional rail*	*Metro*	*Tramway*	*Monorail*	*Cog railway (pure rack system)*	*Funicular*
Track superstructure	Conventional and/or slab track. CWR, UIC 60 Concrete sleepers	Conventional, CWR, UIC 60. Preferably concrete or wooden sleepers	Conventional Slab track	Usually slab track	Usually embedded in the pavement	Single beam in orthogonal profile or of type I from concrete and rarely from steel	Conventional with additional cogged bar in the centre and parallel with the main rails	Conventional or slab track with pulleys along the length of the line to support cable
Maximum transportation work (passengers/ hour/ direction)	High. Demand for trips between terminal stations	Demand for passenger transport between terminal stations as well as intermediate stations	60,000 Without seasonal fluctuations High demand for trips between intermediate stops of the route	45,000	15,000	Small systems: 2,000 Large systems: 12,500 Compact systems: 4,800	Low/medium transportation capacity	Low transportation capacity (500–2,000)
Integration relative to ground surface	Mainly at grade	Mainly at grade	At grade and underground in small parts	Underground integration for the biggest part of the route	At grade for the biggest part of the route	Mainly over ground integration	At grade (very rarely with underground sections)	At grade. Rarely underground
Train formation	4–10 vehicles	4–10 vehicles	2–8 vehicle Push–pull trains	4–10 vehicles	Articulated trains	Articulated trains (2–6 vehicles)	Usually a single railcar	Usually a single car
Rolling stock	EMU and special loco-hauled trains	MU and loco-hauled trains	Usually MUs	Special rolling stock	Special rolling stock low (usually) height floor	Special rolling stock	Electric and diesel railcars	Cabin vehicles (one ascending, one descending)

(*Continued*)

Table 1.6 (Continued) Main technical and operational characteristics of passenger railway transport systems

Technical and operational characteristics	*High-speed interurban rail*	*Conventional interurban rail*	*Suburban/ urban/regional rail*	*Metro*	*Tramway*	*Monorail*	*Cog railway (pure rack system)*	*Funicular*
Signalling	Cab signalling	Electric side/ Automated train protection system	Electric side/ cab signalling/ Automatic train protection system	Cab signalling	Electric side signaling	–	Usually mechanical signalling	–
Level crossings	Prohibited	Permitted	Automatic barriers with warning light and sound signals	Nonexistent	Mandatory	Nonexistent	Permitted	Nonexistent
Environment	Interurban	Interurban	Suburban Periurban Regional	Urban	Urban	Urban. Connections in recreational parks and zoos. Connection of airports with city centres	Mountain Rarely urban. Connecting locations with great altitude difference	Tourist connections, urban connections with great altitude differences
Maximum running speed (km/h)	200–320(350)	<200	120–160	90–100	80–90	60–90	40 (usually 15–25)	50 (Constant speed, usually < 20)
Implementation cost (€ M/ track km)	10–40 (usually 15–25) (Infrastructure only, double track)	8–12 (Infrastructure only, double track)	10–20 (Infrastructure only, double track)	60–130 (Infrastructure and rolling stock, double track	15–35 (Infrastructure and rolling stock, double track)	30–90 (Infrastructure and rolling stock)	10–15 (Infrastructure only)	15–25 (Infrastructure and rolling stock)
Peculiarities	Very high speed.High implementation cost	Passenger and freight trains usually share the same infrastructure	Very high transportation capacity. Vehicles with many passenger seats	Underground corridor High transportation capacity Very high implementation cost	Specific superstructure Surface integration into urban areas Very sharp horizontal curve radii	Elevated permanent way Specific superstructure Panoramic view	Steep gradient. Rolling stock with effective braking system Specific superstructure Light vehicles	Very steep gradient Short-length connection Movement via cables Limited, low transportation capacity

the other modes. The difficulties inherent to such an approach are numerous, since (a) one must compare competitive modes (with similar functionality) in order for the results to be comparable, (b) in the literature, one comes across statistical data comparing the transport systems at various levels with regard to the geographical presence of the means at a global level, at a continental level, within a country or a group of countries (e.g., EU member states) and so on (c) For the same evaluation indicator, there is a wide variation in the values compared depending on the literature; this shortcoming, which is particularly strong, requires cross-checking data from numerous and various sources in order for the final results to be as reliable as possible.

Within this framework the two cases below are compared:

Case 1: Long-distance trips (S = 500–1,500 km). Aeroplanes and high-speed trains are compared.

Case 2: Urban trips. Metro, tram, urban bus and private car are compared.

The comparison does not include parameters related to the natural environment, which are comprehensively examined in Chapter 19.

1.5.3.1 Comparison of air and high-speed train transport

In order to compare high-speed trains with aeroplanes, eight specific connections in Europe served by both means of transportation (Rome-Naples, Rome-Florence, Madrid-Barcelona, Madrid-Seville, London-Paris, Amsterdam-Paris, Brussels-Amsterdam, Paris-Lyon) have been taken into consideration (Pyrgidis and Karlaftis, 2010). These routes either concern domestic connections or international connections. The routes concern the 2009–2010 period.

After analysing the data, the following may be reasonably concluded:

- Regarding travel times, aeroplane is the fastest mode (ratio 1:1.7 for short distances (250 km), 1:3 for long distances (500 km)).
- Regarding transfer times, the aeroplane prevails only for long and very long distances.
- The ratio of number of daily services between the aeroplane and the high-speed train is 1:4.
- Travelling is clearly less expensive by train than it is by aeroplane.
- Transport capacity is calculated by multiplying the number of journeys carried out daily in the eight connections examined by the number of passengers who can be carried every day by each mode. Specifically regarding air travel, an average aeroplane load is equal to 247 passengers. For connections over 400 km, the conclusion to be drawn is that the transport capacity ratio between aeroplane and high-speed train is 1:3.

1.5.3.2 Comparison of urban systems

Table 1.7 shows the comparison between different urban transport systems, in terms of transportation capacity, commercial speed, fares, frequency and space occupied.

1.6 HISTORICAL OVERVIEW OF THE RAILWAY AND FUTURE PERSPECTIVES

The development of the railway is directly related to the use of steam as a source of energy and the exploitation of coal and iron mines (Profillidis, 2014). Table 1.8 shows the milestones in the history of railways.

Table 1.7 Comparison of urban transport systems

Comparable parameters	*Comparable systems*		
Max number of passengers transported per hour per direction	Metro/tram 2.5:1	Metro/urban bus 13:1	Tram/urban bus 5:1
Commercial speed	Urban bus/tram/private car/metro 1:1.5:2.0:2.5		
Fare	Metro/tram/urban bus 1:1:1 Metro/private car peak hours 1:15 – off-peak hours 1:5		
Headway between vehicles	Metro/tram/urban bus 1:2:3		
m^2 of road occupied per transported passenger	Tram/urban bus/private car 1.2:1.9:23.7–40		

The year 1825 is considered to be the starting point in the history of railway, and George Stephenson, an English engineer, is considered as its pioneering figure.

The railway is perhaps the only technology that during its course of development reached an early peak, then found itself being in question and during the last decade it not only managed to rise again but also to be at the cutting edge of technology in many countries.

In short, the railway dominated terrestrial transportation for over 100 years (1830–1950). During this period, it made an enormous contribution to transportation and civilisation. The railway is considered to be the mode of transport that laid down the foundations for inland development on all continents. If it were not for the railway, the coastal towns would have become powerful, as they would be the only ones able to support the growing demand for transport of goods by using the basic means of mass transportation, that is, ships. The 1950s saw the start of the struggle between the railway on the one hand and the aeroplane and car on the other, a struggle which was to intensify from the mid-1960s onwards. At the dawn of the 1970s, not only had the railway begun to lose a worrying amount of ground in terms of its share of the transport market, but also railway organisations begun to suffer financially (loans, deficits) and to be significantly dependent on state budgets, having problematical cross-border services.

Table 1.8 Important milestones in the history of the railway

1800	Discovery of the steam engine (Watt)
1822	Operation of the first factory to construct steam engines (Stephenson, Newcastle, England)
1825	First commercial steam-powered railway journey (Stockton–Darlington line, England)
1830	Operation of first passenger steam-powered railway (Liverpool–Manchester)
1830	First flat-bottom rails (Stevens, USA)
1858	First steel wheels (Bessemer)
1879	Unveiling of first electric locomotive (Siemens-Halske, Germany)
1938	First appearance of diesel traction
1964	Operation of first high-speed train (V_{max} = 210 km/h, Japan, Tokyo–Osaka line)
1981	Operation of first high-speed train in Europe (V_{max} = 260 km/h, France, TGV PSE)
1989	Operation of speed train 300 km/h (France, TGV-A)
1990	First conventional train to achieve a speed in excess of 500 km/h, (515.3 km/h, France)
2007	The most recent record speed for a conventional train (574.8 km/h, France)
2009	Operation of super-fast trains V_{max} = 350 km/h (China)

According to the European Union's institutions that are responsible for the drawing up and implementation of transport policies, 'competition' has been the solution to overcome the economic impasse afflicting the rail sector in recent years. For this purpose, important amounts were and are still being spent for the construction of new railway infrastructures and modern rolling stock, the research for increasing running speeds was intensified and crowned with success, whereas significant efforts were made to improve the services offered in both freight and passenger transport.

In parallel, while maintaining all its intrinsic advantages (safety, great transport capacity, environmental friendliness, etc.), rail transport had to try to operate with more flexible organisational structures, and the monopoly had to be lifted.

Within this framework, the European Commission published a series of directives calling for the revitalisation of the railway sector and the increase of its competitiveness.

The first attempt to draw up railway legislation was made in 1991 with the adoption of Directive 91/440 by the Council of Ministers. This Directive, concerning the development of the railway in the community, is the first attempt to open up railway transportation to competition. For the first time it introduced the right of free access both for international railway consortiums and companies carrying out combined transportation to the rail infrastructure. The provisions of the Directive also imposed the accounting separation between infrastructure activities and operation activities.

Since 2010, the entire rail services within the European Community Railway Network are open.

Over the last 30 years, the improvement in the quality of life in the large cities, the dramatic rise in road and airport congestion, the intensification of air and noise pollution, as well as the continuing energy crisis, have all created a massive ecological issue. Thus, the railway has made a comeback since it is an ecologically friendly mode of transport, and has become more up-to-date, and can move at very high speeds. The use of rail transportation is judged more and more to be imperative, both for movement within urban and suburban environments, and also in order to serve the needs for long-distance traveling.

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Chapter 2

Loads on track

2.1 CLASSIFICATION OF LOADS

The railway track is subjected to loads that are vertical, transversal and longitudinal (Figure 2.1). Apart from the forces that may be exerted in case of an earthquake, all other forces are generated by the rolling stock which is running on the track (traffic loads).

The vertical loads are exerted on the rail rolling surface and are transferred to the subgrade through the various components of the track. During their transfer, the surface area of exertion of the internal forces increases, while the developing stresses decrease (Esveld, 2001; Lichtberger, 2005).

The transversal loads are first transferred by the wheels to the rails either solely through the rail rolling surface (when there is no flange contact) or through the wheels' flanges (when there is contact). Further on, the loads are transferred through the components of the track panel (fastenings, elastic pads, sleepers) to the track bed layers.

The longitudinal loads are exerted on the rail rolling surface and, similar to the transversal loads, they are transferred to the track bed layers. They are distributed to a larger number of sleepers compared with the vertical loads.

Depending on their nature, track loads are classified as follows:

- *Static loads*: As a result of the gross weight of the rolling stock. They are exerted on the track by the rolling stock permanently, whether the rolling stock is immobilised or running.
- *Semi-static or quasi-static loads*: They are exerted on the rolling stock through which they are transmitted to the track for a given period of time after which they vanish, as soon as the cause provoking such loads stops existing. Loads developed as a result of crosswinds and as a result of the residual centrifugal force are examples of semi-static loads.
- *Dynamic loads*: They are caused as a result of
  - Track defects and the heterogeneous vertical stiffness of the track
  - Discontinuities of the rolling surface (at joints, switches, etc.)
  - Wear on the rail rolling surface and on the wheel treads
  - The suspension system of the vehicles and the asymmetries of the rolling stock

They vary in relation with time and cause oscillations of different parts of the vehicle.

Table 2.1 presents the classification of loads applied on the track according to the direction in which they act upon the track, and provides information regarding their origin and their magnitude (Pyrgidis and Iwnicki, 2006; Pyrgidis, 2009).

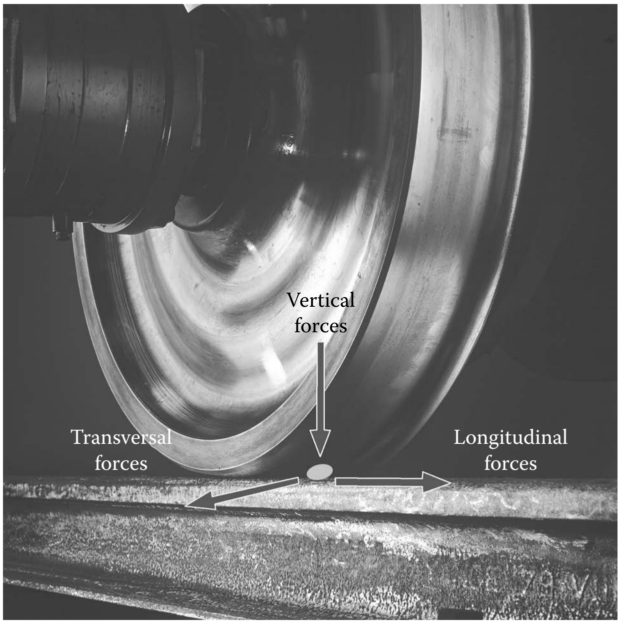


Figure 2.1 Forces acting on the rail in all three directions. (Adapted from Urtado, M. 1993, SNCF Médiathèque, France.)

2.2 VERTICAL LOADS ON TRACK

Vertical loads play a decisive role in the design, construction, operation and the maintenance of the track. They are a part of the mathematical expressions of all lateral and longitudinal forces acting on the track, either directly or indirectly. More specifically, the vertical strain will determine the selection of the type of the rail, the material of the sleepers and the distance between them, the fastenings as well as the dimensioning of the elastic pads and the track bed layers.

Last but not least, vertical loads play an important role in the growth rate of deterioration of the track and the damage of the rolling stock.

As already outlined in Section 2.1, the vertical loads on track are divided into static, quasi-static and dynamic. This classification is consistent in all international literature (Alias, 1977; Esveld, 2001). Apart from these three categories it is also useful to include and examine separately an additional category of vertical loads which can be called 'characteristic loads'. Included in this category are

- The axle load Q
- The total daily traffic load T_f
- The design vertical wheel load Q_d
- The design loads of bridges

A special characteristic of all the above loads, which can be static or dynamic, is that they determine the dimensioning of the railway track as well as the maintenance policy to be followed to a great extent.

During the recent years, a continuous increase of the value of the vertical rail loads has been observed. This appears as an increase of the axle load, of the length and, consequently,

Table 2.1 Loads applied on track

Direction of forces as to the rail rolling surface plane	*Name – symbol of loads*	*Type of effort*	*Cause of generation*	*Analytical expression (j = 1, wheel 1) (j = 2, wheel 2)*
Vertical loads	Vertical axle load Q (characteristic load)	Static	Vehicle mass	$Q = \left(\frac{\overline{M}}{4} + \frac{M'}{2} + m\right) \cdot g$ (vehicle with 2-axle bogies)
	Vertical wheel load Q_o	Static	Vehicle mass	$Q_0 = \frac{1}{2} \cdot \left(\frac{\overline{M}}{4} + \frac{M'}{2} + m\right) \cdot g$ (vehicle with 2-axle bogies)
	Vertical load due to crosswinds Q_w	Quasi-static	Crosswinds	$\pm Q_{wj} = H_w \cdot \frac{q_o}{2 \cdot e_o}$
	Vertical load due to residual centrifugal force Q_{nc}	Quasi-static	Cant deficiency at the curvatures of track in the horizontal alignment	$\pm Q_{ncj} = F_{nc} \cdot \frac{h_{KB}}{2 \cdot e_o} = Q \cdot \frac{1 \cdot h_{KB}}{4 \cdot e_o^2}$
	Dynamic vertical wheel load Q_{dyn}	Dynamic	Track defects, wear of the rail rolling surface, discontinuities on the running rolling surface, asymmetries in rolling stock	$Q_{dynj} = Q_{dyn1j} + Q_{dyn2j} + Q_{dyn3j} + Q_{dyn4j}$
	Total vertical wheel load Q_t	Static, quasi-static, dynamic	Total amount of vertical loads applied	$Q_{tj} = Q_{oj} \pm Q_{wj} \pm Q_{ncj} + Q_{dynj}$
	Design vertical wheel load Q_d (characteristic load)	Static, quasi-static, dynamic	Augmented total vertical wheel load Q_t	$Q_{dj} = Q_{oj} \pm Q_{ncj} \pm Q_{wj} + 5 \cdot [\sigma(Q_{dyn3_j})^2 + \sigma(Q_{dyn1_j} + Q_{dyn2_j})^2]^{1/2}$
Vertical loads (continued)	Design loads of bridges (characteristic loads)	Static and dynamic	Static and dynamic loads	Loading model 71, load models SW/0 and SW/2 (heavy traffic)
	Total daily traffic load T_f (characteristic load)	Static	Daily load of all passing traffic on the line	$T_f = T_p \cdot \frac{V_{max}}{100} + T_g \cdot \frac{Q_{Do}}{18 \cdot D_o}$
Transversal loads	Total gravitational force (or restoring force or gravitational stiffness) S_p	Dynamic	Wheel profile, rail head profile	$S_p = 2 \cdot Q_o \cdot \frac{1}{R - R'} \cdot y = 2 \cdot Q_o \cdot \frac{\gamma_e \cdot y}{R \cdot \gamma_o}$

(*Continued*)

Table 2.1 (Continued) Loads applied on track

Direction of forces as to the rail rolling surface plane	*Name – symbol of loads*	*Type of effort*	*Cause of generation*	*Analytical expression ($j = 1$, wheel 1) ($j = 2$, wheel 2)*
	Lateral creep forces T	Dynamic	Wheel profile, rigid wheel linkage, lateral creep phenomena, when the rolling direction of the wheelset forms an angle with its direction of displacement	Straight path $T_1 = T_2 = -c_{22} \cdot \left(\frac{y'}{V} - \alpha\right)$ Curves $T_1 = T_2 = -c_{22} \cdot (-\alpha)$
	Longitudinal creep forces X	Dynamic	Wheel profile, rigid wheel linkage, longitudinal creep phenomena, when the rolling direction of the wheelset forms an angle with its direction of displacement	Straight path $X_1 = -c_{11} \cdot \left(\frac{x'}{V} - \frac{e_o}{V} \cdot \alpha' - \frac{\gamma_e}{r_o} \cdot y\right)$ $X_2 = -c_{11} \cdot \left(\frac{x'}{V} + \frac{e_o}{V} \cdot \alpha' + \frac{\gamma_e}{r_o} \cdot y\right)$ Curves $X_1 = -c_{11} \cdot \left(-\frac{\gamma_e}{r_o} \cdot y + \frac{e_o}{R_c}\right)$ $X_2 = -c_{11} \cdot \left(-\frac{\gamma_e}{r_o} \cdot y - \frac{e_o}{R_c}\right)$
Transversal loads (continued)	Crosswind force H_w	Quasi-static	Crosswinds	H_w
	Residual centrifugal force F_{nc}	Quasi-static	Cant deficiency at the curvatures of track in the horizontal alignment	$F_{nc} = \frac{Q}{g} \cdot \left(\frac{V^2}{R_c} - g \cdot \frac{U}{2 \cdot e_o}\right) = \frac{Q \cdot I}{2 \cdot e_o}$
	Guidance force F	Dynamic	Flange contact ($y = \sigma$)	F_j
	Force due to vehicle oscillations P_{dyn}	Dynamic	Oscillation of car body and bogies	P_{dyn}

(*Continued*)

Table 2.1 (Continued) Loads applied on track

Direction of forces as to the rail rolling surface plane	*Name – symbol of loads*	*Type of effort*	*Cause of generation*	*Analytical expression ($j = 1$, wheel 1) ($j = 2$, wheel 2)*
	Total transversal force ΣY	Quasi − static + Dynamic	Algebraic sum of all transversal forces exerted upon the track	$\Sigma Y = \pm F_j \pm (T_1 + T_2) \pm H_w \pm S_p \pm P_{dyn} \pm F_{nc}$
Longitudinal (horizontal) loads	Rail creep forces	Dynamic	Trains moving downhill	
	Adhesion force Π	Static	Wheel–rail contact	$\Pi = \mu \cdot Q_o$ (power wheels)
	Traction effort on the treads F_t	Static	Train traction	$F_t = P_t \cdot V$
	Temperature forces N	Static	Temperature changes	$N = \pm E \cdot A_r \cdot \alpha_t \cdot \Delta_t$
	Braking forces N_{br}	Static	Vehicle braking	$N_{br} = 0.25 \cdot \Sigma Q$
	Acceleration forces N_{ac}	Static	Vehicle acceleration	N_{ac}
	Fishplate forces P_f	Dynamic	Passage of trains	$P_f = Q_o + 2 \cdot \alpha_f \cdot V_P \cdot \sqrt{k \cdot USM}$

Source: Adapted from Pyrgidis, C. and Iwnicki, S. 2006, *International Seminar Notes*, EURNEX-HIT, Thessaloniki; Pyrgidis, C. 2009, Transversal and longitudinal forces exerted on the track – Problems and solutions, *10th International Congress Railway Engineering* – 2009, 24–25 June, 2009, London, Congress Proceedings, CD.

of the weight of the trains, of the daily traffic load and finally, as a result of the significant increase of the train speeds (resulting in an increase of the dynamic vertical loads), respectively (Riesberger, 2008).

2.2.1 Static vertical loads

2.2.1.1 Axle load

The term axle weight or axle load describes the static load Q which is individually transferred by each axle of a vehicle and in general of a train, through the wheels to the rails. The axle load is classified as 'characteristic load'.

Considering a symmetric loading of the various vehicle parts, the axle load substantially expresses the quotient of the total vehicle weight divided by the total number of axles.

For example, in the case of a vehicle with 2-axle bogies, the following mathematical equation applies:

$$Q = \left(\frac{\bar{M}}{4} + \frac{M'}{2} + m \right) \cdot g \tag{2.1}$$

where

Q: axle load
$\bar{M}$: car body mass
M′: mass of one bogie
m: mass of one railway wheelset (axle + wheels + axle boxes)
g: gravity acceleration

The axle load is indirectly or directly involved in the analytical expressions of all the forces applied on the wheel–rail contact surface and affects the behaviour of both the rolling stock and the track. Especially for track run by very heavy vehicles, an increase in the number and size of track defects is observed as well as a fatigue of the track superstructure materials leading to an increase of the track maintenance needs and cost.

The International Union of Railways (UIC) classifies the tracks depending on the maximum permitted axle load into four categories: A, B, C and D (Table 2.2).

Each category is divided into subcategories depending on the distributed load per metre of length (total vehicle load divided by the free length between buffers).

Table 2.2 Categories of tracks according to the permitted axle load of trains in traffic (in accordance with UIC)

Track category	*Axle load (t)*
A	16
B	18
C	20
D	22.5

Source: Adapted from UIC. 1989, Fiche 714R, Classification des voies des lignes au point de vue de la maintenance de la voie.

The maximum axle load a track can support is a function of all the parameters involved in the construction of the track superstructure and substructure. The maximum axle load differs from country to country and in most countries, from track to track.

The increase of the track gauge allows a significant increase of the axle load.

Axle loads $Q > 16–17$ t are considered prohibitive for the development of very high speeds ($V \geq 250$ km/h).

The axle load is characterised as an important issue in order to ensure railway interoperability.

2.2.1.2 Wheel weight

The term 'wheel weight' or 'wheel load' refers to the static load Q_o which is individually transferred by each wheel of the vehicle to the corresponding rail.

Considering a symmetric loading of the vehicle, the following mathematical equation applies:

$$Q_o = \frac{Q}{2} \tag{2.2}$$

In practice, the loads of both wheels of each axle, especially in the case of running in curves, are not equal to each other.

The wheel weight and, in particular, the weight distribution to the two wheels is directly linked to the phenomena of derailment and overturning of vehicles.

2.2.1.3 Daily traffic load

The qualitative and quantitative assessment of track traffic is usually expressed by the total daily traffic load T_f (in t). The load T_f is classified as the 'characteristic load'.

On the basis of the value of the total daily traffic load, the tracks are classified into categories for the ultimate purpose of standardising the track dimension and maintenance.

To calculate T_f the following two mathematical equations have been suggested by the International Union of Railways:

$$T_f = T_p \cdot \frac{V_{max}}{100} + T_g \cdot \frac{Q_{D_o}}{18 \cdot D_o} \tag{2.3}$$

where

T_f: total daily traffic load (in t)
T_p: daily traffic load of passenger trains (in t)
T_g: daily traffic load of freight trains (in t)
V_{max}: maximum running speed (in km/h)
D_o: minimum wheel diameter of running trains along the line (in m)
Q_{Do}: maximum passing axle load (wheels of diameter D_o) (in t)

On the basis of Equation 2.3 the tracks are classified into four categories (Table 2.3).

$$T_f = S_V(T_V + K_t T_{tv}) + S_m(K_m T_m + K_t T_{tm}) \tag{2.4}$$

where

T_f: total daily traffic load (in t)
T_V: average daily traffic load of trailer passenger cars (in t)
T_m: average daily traffic load of trailer freight wagons (in t)

Table 2.3 Classification of tracks based on relation (2.3)

Track category	*Total daily traffic load (t)*
I	$T_f > 40{,}000$
II	$40{,}000 \geq T_f > 20{,}000$
III	$20{,}000 \geq T_f > 10{,}000$
IV	$T_f \leq 10{,}000$

T_{tv}: average daily traffic load of passenger trains' power vehicles
T_{tm}: average daily traffic load of freight trains' power vehicles (in t)
K_m: coefficient with values varying between 1.15 (standard value) and 1.45 (when >50% of the traffic takes place with vehicles of axle load Q = 22.5 t or when 75% of the traffic takes place with vehicles of axle load Q ≥ 20 t)
K_t: coefficient that depends on the rolling conditions of the power vehicle axles on the track. It is usually equal to 1.40
S_v, S_m: coefficients with values depending on the speed of passenger (with the highest speed) and freight (with the lowest speed) trains, respectively, running on the track

On the basis of Equation 2.4 the tracks are classified into six groups according to the UIC (Table 2.4) (UIC, 1989).

2.2.2 Quasi-static vertical loads

2.2.2.1 Vertical wheel load due to crosswinds

The vertical load Q_w due to crosswinds is described by the following mathematical equation (Figure 2.2) (Esveld, 2001):

$$\pm Q_w = H_w \frac{q_o}{2e_o} \tag{2.5}$$

where
H_w: crosswind force applied on the geometrical centre of the lateral surface of the car body
q_o: vertical distance between the geometrical centre of the lateral surface of the car body and the rail rolling surface
$2e_o$: distance between the vertical axis of symmetry of the two rails

Table 2.4 Classification of tracks based on relation 2.4

Track category	*Total daily traffic load (T_f)*
UIC 1	$T_f > 130{,}000$ t
UIC 2	$80{,}000 \text{ t} < T_f \leq 130{,}000$ t
UIC 3	$40{,}000 \text{ t} < T_f \leq 80{,}000$ t
UIC 4	$20{,}000 \text{ t} < T_f \leq 40{,}000$ t
UIC 5	$5{,}000 \text{ t} < T_f \leq 20{,}000$ t
UIC 6	$T_f \leq 5{,}000$ t

Source: Adapted from UIC. 1989, Fiche 714R, Classification des voies des lignes au point de vue de la maintenance de la voie.

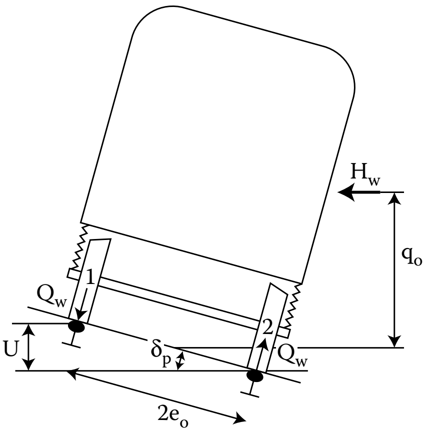


Figure 2.2 Vertical wheel load $\pm Q_w$ due to crosswinds – motion in curve. (Adapted from Esveld, C. 2001, *Modern Railway Track*, 2nd edition, MRT-Productions, West Germany.)

When the force H_w is directed from wheel 2 toward wheel 1, then wheel 1 load increases by Q_w, while wheel 2 load decreases by the same value.

As a result of crosswinds, the load is applied during motion both on straight path and in curves and stops once the wind loads disappear.

2.2.2.2 Vertical wheel load due to residual centrifugal force

The vertical load Q_{nc}, which is due to residual centrifugal force F_{nc}, is expressed as (Figure 2.3)

$$\pm Q_{nc} = F_{nc} \frac{h_{KB}}{2e_0} = Q \frac{I\, h_{KB}}{4e_0^2} \tag{2.6}$$

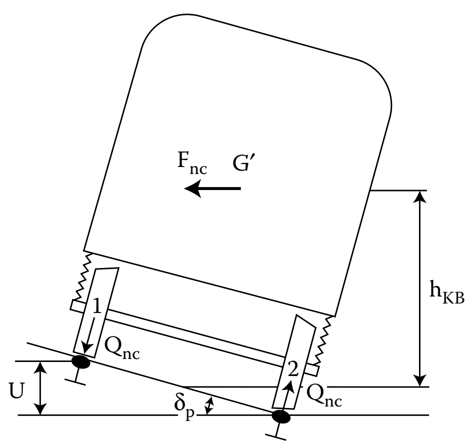


Figure 2.3 Vertical wheel load $\pm Q_{nc}$ due to residual centrifugal force. (Adapted from Esveld, C. 2001, *Modern Railway Track*, 2nd edition, MRT-Productions, West Germany.)

where

F_{nc}: residual centrifugal force
I: cant deficiency
h_{KB}: distance between the vehicle's centre of gravity G′ and the rail rolling surface
U: track cant
R_c: radius of curvature in the horizontal alignment
δ_p: angle of cant

The load which is due to the residual centrifugal force is only applied during motion at curved segments of the track and its measure is usually 10%–25% of the value of the static wheel load. The load of the wheel rolling on the outer rail is increased by Q_{nc} while the load of the inner wheel is, respectively, decreased by the same value.

2.2.3 Dynamic vertical loads

2.2.3.1 Dynamic vertical wheel load

The total dynamic vertical wheel load Q_{dyn} is calculated as the sum of the four separate dynamic loads, namely:

Q_{dyn1}, Q_{dyn2}, Q_{dyn3}, Q_{dyn4}

where:

Q_{dyn1}: is due to the vehicle's sprung masses
Q_{dyn2}: is due to the vehicle's semi-sprung masses
Q_{dyn3}: is due to the vehicle's unsprung masses
Q_{dyn4}: is due to the oscillations of the elastic parts of the rail-sleeper fixing system

The dynamic forces that are exerted on the track due to the interaction with the rolling stock are random. By employing the linearity hypothesis, their study can be conducted in different frequency ranges, depending on the precise mechanism of oscillation, the causes of the oscillation and also depending on the body that participates in the motion.

At low frequencies (0–40 Hz) interest is focused on the interaction of the total of inert and elastic elements of the vehicle, that is, the unsprung masses (wheelsets), the semi-sprung masses (bogies), the sprung masses (car body) and the primary/secondary suspension with the railway track.

At this frequency range, the track appears to be particularly rigid in relation with the vehicle. As a result it is not taken into consideration during the dynamic study of the mechanical system. The generic mechanism of the oscillations at this particular frequency range mainly consists of the medium and long wavelength track defects L_w, that is, 3–25 m and 25–70 m or 25–120 m, respectively, depending on the speed of the vehicle, a parameter which determines the frequency of oscillation f ($f = V/L_w$, in Hz).

Similarly, at medium and high frequencies (40–400 and 400–2,000 Hz), the interest is focused on the interaction between the unsprung masses of the vehicle (wheelsets) and the track. In this case the vehicle's suspension isolates its total of masses that are capable of contributing to the motion, with the exception of the unsprung masses. As concerns the railway track, when examining the system all components are taken into consideration.

The rails' geometric defects as well as any discontinuities of their running surface are considered to be the generic mechanism for the above frequency range (Giannakos, 2002).

- *Forces at frequency range of 0.5–5 Hz*
  Such forces are due to the sprung masses of the vehicle (car body).
  The vertical accelerations of the car body increase less rapidly than the speed. As a result, the increase of the static vertical wheel load caused by these forces is relatively small for both low and high speeds.
  These loads may be reduced by reducing the natural frequency of the car body or by an improvement of the track's quality.
- *Forces at frequency range of 5–20 Hz*
  Such forces are due to the semi-sprung masses of the vehicle (bogies).
  The vertical accelerations of the bogies increase with speed, yet at a quicker rate than that of the car body.
  In this case the increase of the static vertical load is greater.
  Q_{dyn2} loads may be reduced by lightening the bogies, by reducing the vertical stiffness of the primary suspension springs and by increasing the bogie–axles damping coefficient.
  The presence of continuous welded rails limits the oscillations of the semi-sprung masses.
- *Forces at frequency range of 20–200 Hz*
  Such forces are due to unsprung masses of the vehicle (wheelsets) and the rails.
  The reduction of the axle mass, the reduction of the track stiffness and the increase of the ballast thickness contribute to the reduction of the dynamic vertical forces.
- *Forces at frequency range of 200–2,000 Hz*
  Such forces are due to the oscillations of the elastic parts of the rail–sleeper fixing system (elastic pads, fastenings). They generate noise and wear on the rail rolling surface. They are the main cause of short pitch corrugation on the rails.
  Thus, the total dynamic vertical wheel load Q_{dyn} is the result of adding all the above forces:

$$Q_{dynj} = Q_{dyn1j} + Q_{dyn2j} + Q_{dyn3j} + Q_{dyn4j} \tag{2.7}$$

where
$j = 1,2$: index related to the two wheels of the same wheelset

The very high speeds, the continuously increasing weight and the stiffness of the track's components result in an increase of the effect of the dynamic phenomena and an increase of loads exerted on the track's superstructure, the subgrade and the vehicle.

The calculation of the dynamic forces remains extremely complex, and in some instances it is not possible at all. Most analyses are restricted to quasi-static speculations. In most cases a purely empirical approach based on measurements is adopted.

2.2.3.2 Total vertical wheel load

The total vertical wheel load Q_t is calculated as the sum of all the static, quasi-static and dynamic vertical loads transferred to each wheel by the rolling stock:

$$Q_{tj} = Q_{oj} \pm Q_{wj} \pm Q_{ncj} + Q_{dynj} \tag{2.8}$$

2.2.3.3 Design vertical wheel load

The term 'design vertical wheel load' Q_d refers to the characteristic value of the wheel's vertical load exerted on the railway track, which covers the maximum possible theoretical probability of exceeding such load during the life cycle of the railway track. This probability allows for taking into consideration exceptional track loading conditions, such as, for example, anchors forgotten on the running surface of the rails, rail fragmentation, discontinuities of the rolling surface of the rails, flattening of the wheel treads that exceed the accepted tolerances, etc.

Given the random nature of the load mechanism of the railway track, the probability of the appearance of extreme, maximum wheel loads is generally equal to values around 10^{-6}, that is, one in a 1,000,000.

From all the above, it becomes clear that having an accurate value for the dynamic design load of the wheel is of great significance for the railway track. The probability approach adopted for this calculation is generally based on the increase of the mean value of the total vertical wheel load, which allows to achieve the desired statistical level of safety.

The design vertical wheel load Q_d is considered to be equal to the sum of the static wheel load, the quasi-static wheel load and the mean of the square deviation of the standard deviations of the dynamic forces of the unsprung, sprung/semi-sprung vehicle masses, increased so as to cover the statistical probability of exceeding the calculated load in real conditions (Esveld, 2001; Giannakos, 2002).

Thus, the following mathematical equation applies:

$$Q_{dj} = Q_{oj} + Q_{Hj} + n_p \times [\sigma \times (Q_{dyn3j})^2 + \sigma \times (Q_{dyn1j} + Q_{dyn2j})^2]^{1/2} \tag{2.9}$$

where

- Q_d: design vertical wheel load
- Q_o: static vertical wheel load
- Q_H: quasi-static vertical wheel load
- $\sigma(Q_{dyn3})$: typical deviation of the vertical dynamic forces of the unsprung masses of the vehicle
- $\sigma(Q_{dyn1,}\ Q_{dyn2})$: typical deviation of the vertical dynamic forces of the sprung and semi-sprung masses of the vehicle
- n_p: coefficient of probability augmentation of the mean square value of standard deviations of vertical dynamic forces of a vehicle, taking a value equal to 5.00 (Demiridis and Pyrgidis, 2010)
- j = 1,2: index related to the two wheels of the same wheelset

2.2.3.4 Design loads of bridges

The design loads of bridges are classified as vertical, longitudinal and transversal. Taking into account that vertical loads are the most important ones, all three categories are taken into consideration when studying the vertical forces.

To calculate the design loads of bridges two approaches may be applied (UIC, 1979, 2003; EN 1991-2, 2003a; EN 1993-2, 2006; Gerard, 2003; Tschumi, 2008):

- The static analysis
- The dynamic analysis

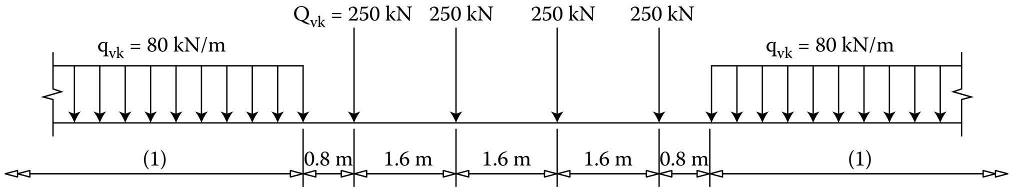


Figure 2.4 Loading model 71. (Adapted from UIC. 2003, 702 Loading diagram to be taken into consideration for the calculation of rail carrying structures on lines used by international services, Technical Report, International Union of Railways, Leaflet.)

2.2.3.4.1 Vertical static loads

The vertical loads can be defined with the aid of two loading models, of which one represents the conventional traffic loads (loading model 71) and the other represents the heavy loads (SW loading model).

Figure 2.4 illustrates the loading model 71 and the respective characteristic values of the vertical loads. In the case where trains running on the track are heavier or lighter than the loading model 71, these loads are multiplied by a coefficient α_{br}. The values of the coefficient α_{br} are decided by the competent authority and are 0.75–0.83–0.91–1.00–1.10–1.21–1.33. Loads of the loading model SW concerning bridges with continuous spans, centrifugal forces, acceleration and breaking forces as well as random forces should also be multiplied by the same coefficient.

Figure 2.5 illustrates the arrangement of the SW loading model while Table 2.5 summarises the respective characteristic values of vertical loads.

2.2.3.4.2 Vertical dynamic loads

Traffic loads causes dynamic oscillation which results in increased stresses and deformations. The main factors that influence the vertical dynamic behaviour are the following:

- Natural frequency of the structure
- Distance between the railway axles
- Running speed on the bridge
- Damping of the construction

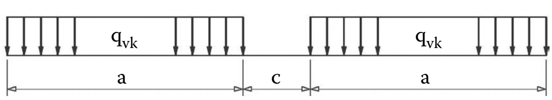


Figure 2.5 Loading model SW. (Adapted from UIC. 2003, 702 Loading diagram to be taken into consideration for the calculation of rail carrying structures on lines used by international services, Technical Report, International Union of Railways, Leaflet.)

Table 2.5 Characteristic values of vertical loads for SW loading models

Load classification	*q_{vk} (kN/m)*	*a (m)*	*c (m)*
SW/0	133	15.0	5.3
SW/2	150	25.0	7.0

- Arrangement of the bridge's deck
- Wheel and rail defects, etc.

To take into consideration the influence of these factors on the dynamic behaviour of bridges, regulation defines the dynamic coefficient φ_{bri} (i = 2 or 3). This coefficient applies only for speeds V ≤ 220 km/h and for natural frequencies which lay within the limits that are set by the regulations. Dynamic coefficient φ_{bri} increases the static stresses and deformations that are caused by traffic loads.

2.2.3.4.3 Transverse loads

- *Centrifugal forces*: Centrifugal forces act at a level of 1.80 m above the rolling surface. These forces are not multiplied by the coefficient φ_{bri}.
- *Total transverse load*: The characteristic value of the lateral force must be considered equal to $\Sigma Y = 100$ kN. This force is not multiplied by the coefficients α_{br} and φ_{bri} and it must always be combined with the vertical load.

2.2.3.4.4 Longitudinal loads

- *Acceleration and breaking forces*: Acceleration and braking forces act on the rail rolling surface in the longitudinal direction of the track and are considered to be uniformly distributed over their length of influence.
  These forces must be combined with the respective vertical loads.

2.2.3.4.5 Other loads

- Aerodynamic effect due to train circulation
- Random effects – derailment
- Earthquake
- Weight, fire, snow, temperature changes, forces developed during construction, forces resulting from collisions and explosions, etc.

Various methods of analysis exist and are proposed (response spectrum method, static equivalent method, etc.).

2.3 TRANSVERSAL LOADS ON TRACK

The transversal forces are directly linked to the safety of the train traffic and the dynamic comfort of the passengers. Under certain circumstances, they may cause the phenomenon of derailment.

They are distinguished into forces provoked by the wheel–rail interaction and forces due to other causes, external to the railway system, such as crosswind, movement in curves where there is either excess or deficiency of cant, etc.

The first category comprises the gravitational forces, the guidance forces, the creep forces and the forces due to vehicle oscillations. The creep forces are further distinguished into transversal and longitudinal forces. Given that they are activated simultaneously and that they affect the transversal behaviour of the vehicles, they are both examined under the transversal loads thematic section.

The second category comprises the residual centrifugal force and the crosswind force.

All the aforementioned forces together constitute the total transversal force transferred through the wheels to the rails and from there on to all the other track components.

2.3.1 Gravitational forces

At the wheel–rail contact surface of each wheel the reaction force R_o is analysed into two components, namely Q_o and S_{po} (Figure 2.6). The transversal component S_{po} is defined as gravitational or restoring force or gravitational stiffness. It is exclusively due to the conicity of the wheels and it acts via the axle on the rail rolling surface.

It is considered as a dynamic force and is equal to

$$Q_o \tan \gamma_o$$

where

Q_o: static wheel load

γ_o: angle between tangent plane and horizontal wheelset in central position

In the case of a single wheelset, there are two restoring forces, one for each wheel (S_{pj}, j = 1, 2) (Figure 2.7).

If 'y' symbolises the transversal displacement of the wheelset according to Figure 2.7 the following mathematical equations apply:

$$S_{p1} = Q_1 \tan\gamma_1 = Q_1\gamma_1 \tag{2.10}$$

$$S_{p2} = Q_2 \tan\gamma_2 = Q_2\gamma_2 \tag{2.11}$$

where

Q_1, Q_2: vertical components of the reactions R_1 and R_2 at contact points I_1 and I_2, respectively

γ_1, γ_2: angles formed by the horizontal plane and the tangent planes at contact points I_1 and I_2, respectively (as γ_1, γ_2 are very small quantities, tan $\gamma_1 = \gamma_1$ and tan $\gamma_2 = \gamma_2$ applies)

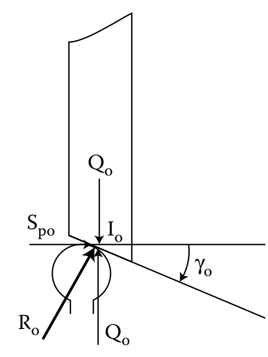


Figure 2.6 Gravitational force per wheel. (Adapted from Pyrgidis, C. 1990, Etude de la stabilité transversale d'un véhicule ferroviaire en alignement et en courbe – Nouvelles technologies des bogies – Etude comparative, Thèse de Doctorat de l' ENPC, Paris.)

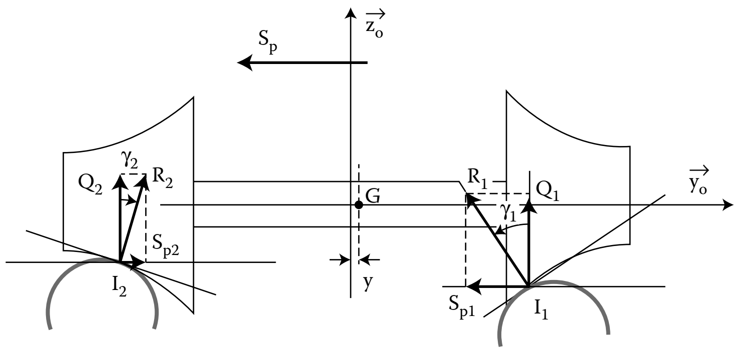


Figure 2.7 Total gravitational force S_p. (Adapted from Pyrgidis, C. 1990, Etude de la stabilité transversale d'un véhicule ferroviaire en alignement et en courbe – Nouvelles technologies des bogies – Etude comparative, Thèse de Doctorat de l' ENPC, Paris.)

From the mathematical resolution of the geometry of the wheel–rail contact and assuming that the wheels are of conical shape with curved slants of constant radius while the heads of the rails are spherical, the following linear relationships are concluded (Pyrgidis, 1990):

$$\gamma_1 = -\gamma_o + \frac{\gamma_e}{R\gamma_o} y \tag{2.12}$$

$$\gamma_2 = -\gamma_o + \frac{\gamma_e}{R\gamma_o} y \tag{2.13}$$

$$\gamma_e = \frac{R \cdot \gamma_o}{R - R'} \tag{2.14}$$

Taking into account mathematical Equations 2.10 through 2.14 and assuming equal weight distribution on each wheel, the following two mathematical expressions for the total gravitational force are concluded:

$$S_p = S_{p1} + S_{p2} = 2 \cdot Q_o \cdot \frac{\gamma_e \cdot y}{R \cdot \gamma_o} \tag{2.15}$$

$$S_p = 2 \cdot Q_o \cdot \frac{1}{(R - R')} \cdot y \tag{2.16}$$

where
R: curvature radius of the wheel tread
γ_e: equivalent (effective) conicity of the wheel
R′: radius of curvature of the rolling surface of the rail head

Considering the mathematical Equations 2.15 and 2.16 we can conclude that the total gravitational force

- Is proportional to the displacement 'y' of the axle's centre of gravity. This means that if for any reason, the axle is displaced laterally then the gravitational force tends to reinstate it to its initial equilibrium position (for $y = 0$, $S_p = 0$).
- Is inversely proportional to the curvature radius R of the wheel tread and proportional to the equivalent conicity γ_e. Thus worn wheels (small R values/big γ_e values), automatically generate a greater gravitational force.

The gravitational force S_p is due to the special profile of the rail wheels (variable conicity). In the case of conical wheels (triangular cross section) the total gravitational force is zero ($R = \infty$, $\gamma_e = \gamma_o$, $\gamma_1 + \gamma_2 = 0$, $S_p = 0$). The same applies in the case of cylindrical wheels (orthogonal cross section) ($R = \infty$, $\gamma_o = 0$, $S_p = 0$).

The presence of the gravitational force does not create problems to any component of the railway system. On the contrary, it plays a balancing role with regard to the forces that tend to disturb the stability of the vehicle wheelsets.

Regarding the track, no wear is caused on the rolling surface of the rails on which it acts.

As regards the rolling stock, the total gravitational force S_p is always desired since it assists in the centring of the railway wheelsets on the track.

Especially in the case where bogies with independently rotating wheels are used (rolling stock of tramway networks), while the longitudinal creep forces become null, the gravitational forces are the only forces enabling the centring of the wheelset on the track (Joly and Pyrgidis, 1996; Pyrgidis, 2004).

On the basis of the above, the increase of the value of the gravitational force leads to an increase of the critical speed* of the vehicle in straight path while it allows for a better geometric positioning of the wheelsets in curves.

Indicatively, when $2Q_o = 13.50$ t, $R = 0.344$ m, $R' = 0.30$ m and $y = 5$ mm then $S_p = 1.53$ t or 15.3 kN.

2.3.2 Creep forces

2.3.2.1 Running on straight path

In the case of conventional axle running on straight path, the analytical expressions of the creep forces resulting from the application of Kalker linear theory are given by the mathematical Equations 2.17 through 2.22 (for the motion parameters, the sign convention adopted in Figure 2.8b applies):

$$X_1 = -c_{11} \cdot \left(\frac{x'}{V} - \frac{e_o}{V} \cdot \alpha' - \frac{\gamma_e}{r_o} \cdot y \right) \tag{2.17}$$

$$X_2 = -c_{11} \cdot \left(\frac{x'}{V} + \frac{e_o}{V} \cdot \alpha' + \frac{\gamma_e}{r_o} \cdot y \right) \tag{2.18}$$

$$T_1 = -c_{22} \cdot \left(\frac{y'}{V} - \alpha \right) - c_{23} \cdot \left(\frac{\alpha'}{V} - \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o} \right) \tag{2.19}$$

* The critical speed V_{cr} of a railway vehicle on a straight path is the speed beyond which the motions of the vehicle bogies become 'unstable' (the sinusoidal motion of the wheelset is no more dampened).

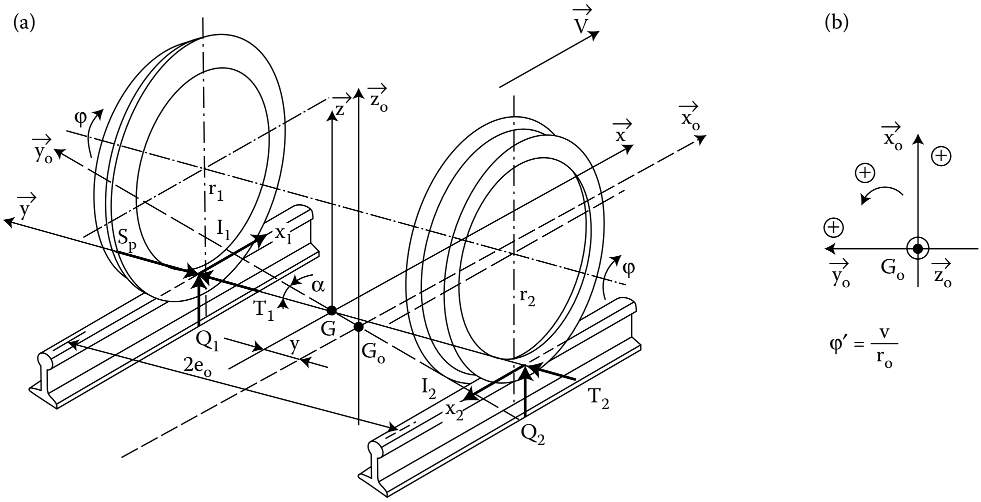


Figure 2.8 Forces applied on the rail-wheelset in random position on the track. (a) Conventional axle running on straight path, (b) sign convention adopted for the motion parameters. (Adapted from Pyrgidis, C. 2009, Transversal and longitudinal forces exerted on the track – Problems and solutions, 10*th International Congress Railway Engineering – 2009*, 24–25 June 2009, London, Congress proceedings, CD; Pyrgidis, C. 1990, Etude de la stabilité transversale d'un véhicule ferroviaire en alignement et en courbe – Nouvelles technologies des bogies – Etude comparative, Thèse de Doctorat de l' ENPC, Paris.)

$$T_2 = -c_{22} \cdot \left(\frac{y'}{V} - \alpha\right) - c_{23} \cdot \left(\frac{\alpha'}{V} + \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right) \tag{2.20}$$

$$M_1 = c_{23} \cdot \left(\frac{y'}{V} - \alpha\right) - c_{33} \cdot \left(\frac{\alpha'}{V} - \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right) \tag{2.21}$$

$$M_2 = -c_{23} \cdot \left(\frac{y'}{V} - \alpha\right) - c_{33} \cdot \left(\frac{\alpha'}{V} + \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right) \tag{2.22}$$

where

X_1, X_2: longitudinal creep forces applied on both wheels
T_1, T_2: lateral creep forces applied on both wheels
M_1, M_2: spin moment on both wheels
x: longitudinal displacement of the wheelset
y: lateral displacement of the wheelset
α: yaw angle of the wheelset
φ: angle of rotation of the wheels and the wheelset
x', y', α', φ': derivative of displacements x, y of the yaw angle α and of angle of rotation φ
c_{11}: longitudinal coefficient of Kalker
c_{22}: transversal coefficient of Kalker
c_{23}, c_{33}: spin coefficients of Kalker

From the mathematical Equations 2.17 through 2.20 the following can be concluded:

- The longitudinal creep forces X_1 and X_2 applied on both wheels create a pair of forces which tend to rotate the railway axle around axis $\vec{z}_o$> (Figure 2.8a). For $x = 0$, the forces X_1 and X_2 are equal in terms of magnitude and opposite in terms of direction.
- The lateral forces applied on each wheel when $C_{23}, C_{33} = 0$ (spin) are equal and act in the same direction.
- The increase of the displacement velocity V reduces the damping terms (x'/V, α'/V, y'/V) that tend to stabilise the axle.
- The increase of the equivalent conicity γ_e and the decrease of the wheel rolling radius r_o lead to an increase of the value of the longitudinal creep forces.

2.3.2.2 Running in curves

In the case of a conventional axle running on a curvature of the track, the analytical expressions of the creep forces resulting from the application of Kalker linear theory are given by the following mathematical equations:

$$X_1 = -c_{11} \cdot \left(-\frac{\gamma_e}{r_o} \cdot y - \frac{e_o}{V} \cdot \alpha' + \frac{e_o}{R_c}\right) \tag{2.23}$$

$$X_2 = -c_{11} \cdot \left(+\frac{\gamma_e}{r_o} \cdot y + \frac{e_o}{V} \cdot \alpha' - \frac{e_o}{R_c}\right) \tag{2.24}$$

$$T_1 = -c_{22} \cdot \left(\frac{y'}{V} - \alpha\right) - c_{23} \cdot \left(\frac{\alpha'}{V} - \frac{1}{R_c} - \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right) \tag{2.25}$$

$$T_2 = -c_{22} \cdot \left(\frac{y'}{V} - \alpha\right) - c_{23} \cdot \left(\frac{\alpha'}{V} - \frac{1}{R_c} + \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right) \tag{2.26}$$

$$M_1 = c_{23} \cdot \left(\frac{y'}{V} - \alpha\right) - c_{33} \cdot \left(\frac{\alpha'}{V} - \frac{\gamma_o}{r_o} - \frac{1}{R_c} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right) \tag{2.27}$$

$$M_2 = -c_{23} \cdot \left(\frac{y'}{V} - \alpha\right) - c_{33} \cdot \left(\frac{\alpha'}{V} + \frac{\gamma_o}{r_o} - \frac{1}{R_c} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right) \tag{2.28}$$

In the case of movement in curves of small radius, with a displacement speed that is approximately equal to the equilibrium speed of the track, the inertia and damping forces may be disregarded when compared with of the elastic forces. Moreover, by ignoring the spin impact the following mathematical equations apply:

$$X_1 = -c_{11} . \left(-\frac{\gamma_e}{r_o} . y + \frac{e_o}{R_c}\right) \tag{2.29}$$

$$X_2 = -c_{11} \cdot \left(+ \frac{\gamma_e}{r_o} \cdot y - \frac{e_o}{R_c} \right) \tag{2.30}$$

$$T_1 = T_2 = -c_{22} \cdot (-\alpha) \tag{2.31}$$

The longitudinal creep forces result in wear of the wheel and rail rolling surface, fatigue of the contact materials and noise.

As shown in Figure 2.8, the longitudinal creep forces result in the horizontal rotation of the axle and along with the lateral creep forces they activate the sinusoidal movement (hunting) of the bogie wheelsets thereby causing oscillations.

The creep forces appear when there is a deviation between the rolling direction of the wheels and the wheelset's direction of displacement. This occurs when there is a transversal displacement y or a yaw angle α of the axle from the initial equilibrium position. Therefore, in order to address creep forces it is required to focus on the parameters that cause such forces.

These factors are the track defects which either preexist due to poor construction of the track or arise during track use. Correcting such track defects essentially involves intervention into the source which produce those creep forces.

Other measures that help reduce the creep forces are the wheel turning at regular intervals, the appropriate choice of the constructional characteristics of the bogies (wheel profile, wheel diameter, stiffness of primary suspension, bogie wheelbase) and the suitable choice of the bogie technology for the operability of the network.

The presence or absence of the longitudinal and lateral component of the creep force depends on how the wheels are connected to the axle while the presence or absence of spin moment depends on the angle formed by the wheel rolling surface plane and the rotation angle.

Table 2.6 presents the forces that are developed on the wheel–rail contact surface for various technologies of railway wheelsets (already manufactured and theoretical) (Frederich, 1985).

To avoid the hunting of the wheelsets, it is essential that the rigid link between the two wheels and the axle be broken. Thus, the two wheels will be able to rotate at different angular velocities while at the same time the following mathematical equation shall remain applicable:

$$\omega_1 r_1 = \omega_2 r_2 = V \tag{2.32}$$

where

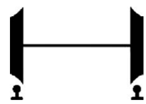
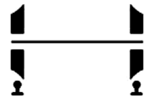
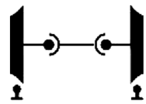
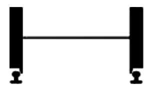
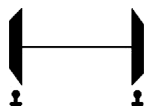
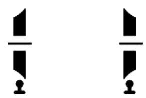
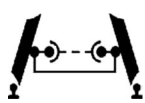
ω_1, ω_2: the angular velocities of the two wheels
r_1, r_2: their rolling radii, respectively

This ensures rolling of the two wheels without creep and elimination of longitudinal creep forces. The technology of bogies with independently rotating wheels is based on this logic.

2.3.3 Crosswind forces

In the case of crosswinds, a transversal force H_w is transferred through the axles to the rail rolling surface. This force is considered as quasi-static and its direction depends on the direction of the wind. In theory, it acts on the geometrical centre of the car body lateral surface.

Table 2.6 Wheelsets technologies: forces exerted on wheels

Technology description	*Schematic representation*	S_p	*T*	*X*	*M*
Conventional axle – wheels of variable conicity		Yes	Yes	Yes	Yes
Axle with independent rotating wheels – wheels of variable conicity		Yes	Yes	No	Yes
Wheels of variable conicity with articulated body axle		Yes	No	Yes	Yes
Conventional axle – cylindrical wheels		No	Yes	Yes	No
Conventional axle – wheels of constant (nonvariable) conicity		No	Yes	Yes	Yes
Independent rotating wheels of variable conicity		Yes	No	No	Yes
Axles with independent rotating inclined wheels of variable conicity		Yes	No	No	No
Conventional axles with inclined wheels of variable conicity		Yes	Yes	Yes	No

Source: Adapted from Frederich, F. 1985, Possibilités inconnues et inutilisées du contact rail-roue, *Rail International*, Brussels, November 1985, 33–40.

The crosswind force H_w is undesirable as it causes an increase of the transversal displacement of the wheelsets and it assists the vehicle's overturning mechanism.

To deal with crosswinds the following measures can be applied:

- Installation of wind barriers
- Speed reduction or the interruption of operation in areas subject to strong crosswinds

2.3.4 Residual centrifugal force

When a vehicle of mass M_t runs at a speed V in a curve where the curvature radius is R_c, then the vehicle's centre of gravity generates the centrifugal force F_{cf}, which pushes the vehicle toward the outer side of the curve (Figure 2.9):

$$F_{cf} = M_t \cdot \frac{V^2}{R_c} \tag{2.33}$$

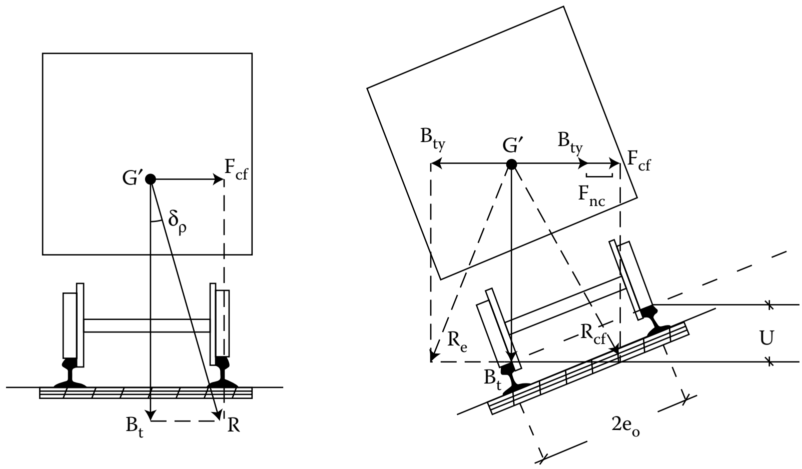


Figure 2.9 Motion in the curvature of the track – residual centrifugal force F_{nc}. (Adapted from Alias, J. 1977, *La voie ferrée*, Eyrolles, Paris.)

Owing to the cant of the track U, the transversal component of the weight B_{ty} is simultaneously applied. B_{ty} acts in a direction opposite to that of the centrifugal force and its value equals

$$B_{ty} = M_t \cdot g \cdot \delta_p = M_t \cdot g \cdot \frac{U}{2 \cdot e_o} \tag{2.34}$$

The difference between forces F_{cf} and B_{ty} expresses the residual centrifugal force F_{nc}. At the level of wheelsets and consequently at the rail rolling surface the following applies

$$F_{nc} = \frac{Q}{g} \cdot \left(\frac{V^2}{R_c} - g \cdot \frac{U}{2 \cdot e_o} \right) = \frac{Q \cdot I}{2 \cdot e_o} \tag{2.35}$$

where
I: cant deficiency

The value

$$\frac{V^2}{R_c} - g\frac{U}{2e_o} \tag{2.36}$$

represents the transversal residual acceleration γ_{nc}.

The increase of the vehicle's speed V as well as the decrease of the radius of curvature R_c and the decrease of the cant U contribute to the increase of the residual centrifugal acceleration.

F_{nc} is considered as a quasi-static force and is always undesirable as it causes not only the displacement of the wheelsets (risk of flange contact), but also problems regarding the transversal dynamic passenger comfort. Moreover, it assists the vehicle's derailment mechanism. However, traffic reasons such as the coexistence of low- and high-speed trains on the same track and the danger of transversal wheelset sliding toward the internal rail render the adoption of a cant which is smaller than the equilibrium cant (theoretical cant) essential, thereby leading to the appearance of a residual centrifugal acceleration.

For example, when Q = 18 t, V = 150 km/h, R_c = 1,500 m, g = 9.81 m/s^2, $2e_o$ = 1.50 m and U = 130 mm, then F_{nc} = 5.6 kN and γ_{nc} = 0.307 m/s^2.

To reduce the residual centrifugal force the following measures can be applied:

- Rational choice of the curve's geometrical data (cant deficiency and cant excess)
- Ergonomic seating design
- Use of tilting trains (see Chapter 13)

2.3.5 Guidance forces

When the transversal displacement 'y' of a railway wheelset is equal to the flange clearance 'σ' between the wheel flange and the rail, the outer side of the wheel flange comes in contact with the inner part of the rails (Figure 2.10).

Transversal dynamic loads are exerted on the contact point and are called guidance forces F_j (j = 1 or 2).

The guidance forces create problems not only to the passengers but also to the rolling stock and the track. More specifically

- They reduce the dynamic passenger comfort (increase of transversal accelerations, jerks)
- They increase the rolling noise considerably
- They cause wear to the wheels and rails
- They increase the fatigue of the bogies
- Under certain circumstances, they may result in a lateral displacement of the track and derailment of the vehicles

The calculation of the guidance forces may be carried out either in practice using dynamometers or in theory (mathematical models of simulation of the vehicle dynamic behaviour).

Therefore, where possible, during railway vehicles' movement the wheel flange contact must be avoided.

To reduce the guidance force the following measures can be taken:

- Appropriate choice of the constructional characteristics of the bogies (wheel profile, wheel diameter, stiffness of bogie primary suspension, bogie wheelbase)
- Appropriate choice of the bogie technology so as to serve network operability

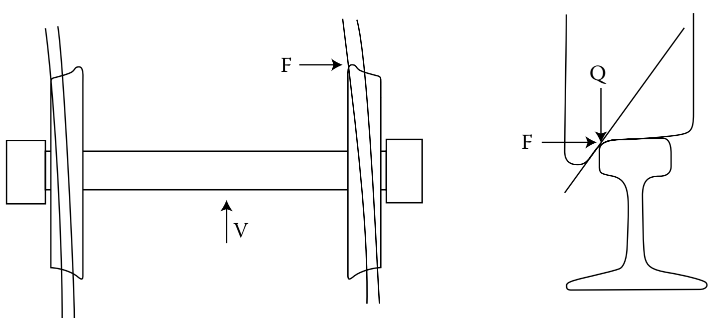


Figure 2.10 Flange contact – guidance force.

- Gauge widening at the narrow curved sections of the track ($R_c < 150–200$ m)
- Rail lubrication at the curved sections of the track

In any case, when there is significant lateral track resistance the safety coefficient increased with regard to the transversal displacement of the track.

2.3.6 Forces due to vehicle oscillations

These transversal dynamic loads P_{dyn} the causes of which are similar to those of vertical dynamic loads confer additional transversal accelerations to the various parts of the vehicle. To address the problem, track defects must be removed and rail grinding must be applied.

2.3.7 Total transversal force

The total transversal force ΣY exerted on the track is equal to the algebraic sum of all transversal forces:

$$\Sigma Y = \pm F_j \pm (T_1 + T_2) \pm H_w \pm S_p \pm P_{dyn} \pm F_{nc} \tag{2.37}$$

Depending on the rolling conditions, some of the terms of the mathematical Equation 2.37 may be null (e.g., for a movement without wheel flange–rail contact, in tangent of track, in the absence of crosswind):

$$\Sigma Y = \pm(T_1 + T_2) \pm S_p \pm P_{dyn}$$

According to Alias (1977) and Profillidis (2014), the total transversal force ΣY is calculated by applying the following empirical formula:

$$\Sigma Y = \frac{QI}{1500} + \frac{QV}{1000} \tag{2.38}$$

where

ΣY: (in t)
I: cant deficiency (in case of movement at curved sections of track) or twist of the track (in case of motion in straight path) (in mm)
Q: axle load (in t)
V: running speed (in km/h)

The first term of the mathematical Equation 2.38 refers to the static forces and the second term refers to the dynamic forces.

To understand better the origin and the operation mechanism of the guidance forces certain basic conditions that influence them, such as the nature of wheel–rail contact and how the loads are transferred from the wheel to the rail need be examined.

Two cases of wheel–rail contact are distinguished, namely (Esveld, 2001; Lichtberger, 2005):

1. *Two-point contact*: When a vehicle moves on a curve and wheels and rails are both new, the wheel–rail contact at the outer rail occurs at two points (Figure 2.11) (Esveld, 2001). This occurs due to the fact that when a train moves on a curve the railway wheelset does not have the freedom to be placed radially. Usually, the front bogie wheelset is

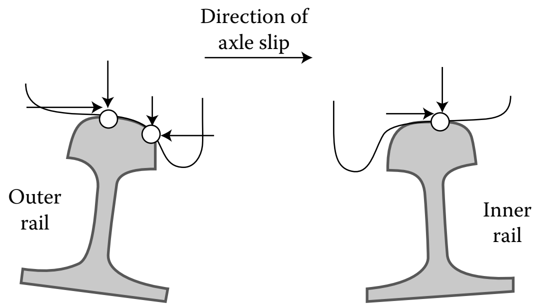


Figure 2.11 Two-point contact. (Adapted from Esveld, C. 2001, *Modern Railway Track*, 2nd edition, MRT-Productions, West Germany.)

displaced toward the outside of the curve. As a result the flange of the outside wheel hits on the inner edge of the outer rail. The generated guidance force forces both of the axle's wheels to slip toward the inside of the track, thereby generating friction forces on both rails as shown in Figure 2.11.

The first point of contact (on the outer rail) lays on the rail surface on which the wheel's tread is rolling, while the second point is the contact point between the wheel flange and the rail. A part of the vertical load is transferred from the wheel to the first point, building up the friction force. On the second contact point the rest of the vertical load and the total transversal force ΣY are transferred.

2. *Single-point contact*: After a certain amount of wear of the wheel, the contact between the wheel and the outer rail occurs at a single point (single-point contact). In this case, the wheel load and the total transversal force ΣY are applied at the same point while friction force builds up as a reaction to the wheel slipping toward the inside of the track.

The nature of the wheel–rail contact on a new rail depends on the radius of curvature in the horizontal alignment. The two-point contact is more likely to occur for a radii between $R_c = 1{,}200$ and 2,000 m. The single-point contact is less likely to occur for radii $R_c < 1{,}200$ m. Finally, multiple-point contact is also less likely to occur for radii $R_c > 2{,}000$ m (Lichtberger, 2005).

2.4 LONGITUDINAL FORCES

2.4.1 Temperature forces

When the rail temperature rises, the rails show a tendency toward an increase or decrease of their length by a value Δl:

$$\Delta l = +\alpha_t \Delta_t l_o \tag{2.39}$$

where
- Δl: Variation of the length of the rail (expansion or contraction) (in mm)
- α_t: Steel thermal expansion coefficient (in grad^{-1})
- Δ_t: $t_{re} - t_{in}$ = actual (recorded) temperature – initial temperature (in °C)
- l_o: Initial rail length (in mm)

This displacement is countered by the friction forces T_{fr} developed between the rails and sleepers and between the sleepers and ballast (Figure 2.12).

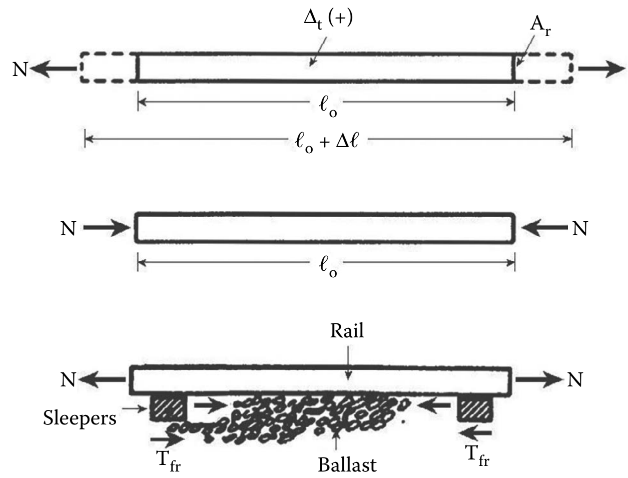


Figure 2.12 Friction forces T_{fr} between the rail and ballast. Temperature forces N. (Adapted from Pyrgidis, C. and Iwnicki, S. 2006, *International Seminar Notes*, EURNEX-HIT, Thessaloniki; Profillidis V. A. 2014, *Railway Management and Engineering*, Ashgate, England.)

Fastenings, sleepers and ballast combined with the rail's weight prevent the rail from expanding or contracting. As a result, the rail is under compressive stress when the temperature increases and under tensile stress when the temperature decreases (Green and Shrubsall, 2002).

In the case of continuous welded rails (CWR) a pair of compressive or tensile forces N applies axially on the rail.

$$N = -E \cdot A_r \cdot \frac{\Delta l}{l_o} = -E \cdot A_r \cdot a_t \cdot \Delta_t \tag{2.40}$$

where
- A_r: rail cross section
- E: steel elasticity modulus

These forces are named temperature forces and are considered to be static forces (Lichtberger, 2005). Their values remain constant almost throughout the length of the CWR. At a distance of approximately $l_A = 150$ m from each edge of the CWR (expansion zone) the value of force N is gradually decreased to zero at both the rail's edges.

When $\Delta_t = 40°C$, $E = 2.1 \times 10^3$ t/cm^2, $\alpha_t = 1.2 \times 10^{-5}$ grad^{-1} and rail UIC 50 ($A_r = 63.93$ cm^2) then the resulting force is N = 64.5 t.

If the track is subject to excessive compressive stress, it shall buckle. For situations where tensile forces are excessive, the rail shall break (Figure 2.13), most likely at a welding point where the rail is usually weaker. Both phenomena are dangerous; however, buckling is particularly dangerous because the deviation of the track from its 'correct' geometric position causes lateral displacement and yaw angle of the wheelsets thus favouring the emergence of creep and guidance forces, whose adverse impacts have already been analysed. The rail temperature for which the total longitudinal force on the rail is null is named rail neutral temperature (RNT). It is often related to the temperature of rail lying.

The temperature at which laying of continuous welded rails happens ensures the development of the lowest possible values of internal stresses (and thus the length of the expansion zone is minimised) is named design rail neutral temperature (DRNT).

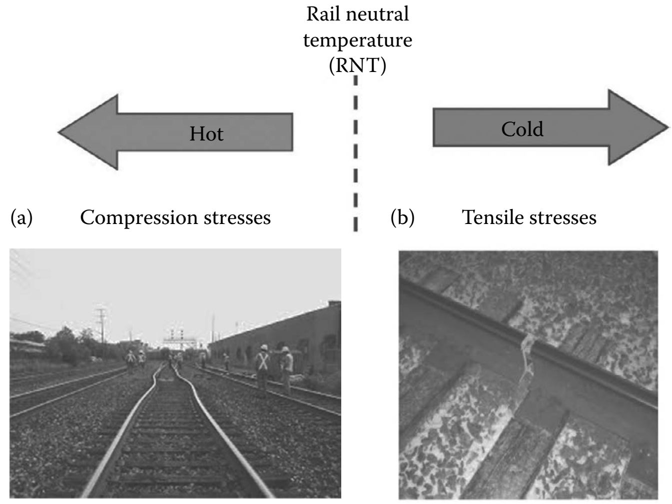


Figure 2.13 Effect of temperature changes on the rails. (a) Adapted from Transportation Safety Board of Canada, 2006, online image available at: http://www.tsb.gc.ca/eng/rapports-reports/rail/2006/r06t0153/r06t0153.asp; (b) Adapted from online image available at: http://www.fuzzyworld3.com/3um/viewtopic.php?f=5&t=4000.)

Track buckling commonly occurs in the transverse direction (Figure 2.13). The activation of lateral buckling effect is facilitated by the rails' transversal defects, the temperature difference between the DRNT and the RNT, the high dynamic train loads and the low lateral track resistance (Lanza Di Scalea, 2012).

For the same track features, at curved segments of the horizontal alignment a lower temperature increase is required for the transversal buckling effect to appear. This is expected because at curves the rail is already curved; hence, its further lateral shift is easier to occur.

Buckling is dealt with by increasing the lateral stiffness of the track (elastic fastenings, concrete sleepers, ballast windrow up to rail height outside the sleeper) and more particularly in the case of continuous welded rails, by stress releasing.

Vertical buckling rarely occurs due to the increased weight of the track's panel.

In the case of bridges, the temperature difference Δ_t provokes direct stressing on the bridge as well as on the track's superstructure components. As already mentioned in Section 2.2.3.4 the forces due to temperature changes are taken into consideration in the design. Furthermore, the position where the breather switches (or expansion joints) are placed is affected by these temperature changes.

2.4.2 Rail creep forces

Rail creep is defined as the gradual displacement of the rails with regard to the sleepers as well as the gradual displacement of the rails and sleepers with regard to the ballast, in the direction of the train's movement.

This specific phenomenon is due to (Australian Rail Track Corporation LTD, 2014)

- The braking of the train, which 'pushes' the rails in the direction of movement.
- The acceleration of trains, which 'pulls' the rails in the opposite direction.

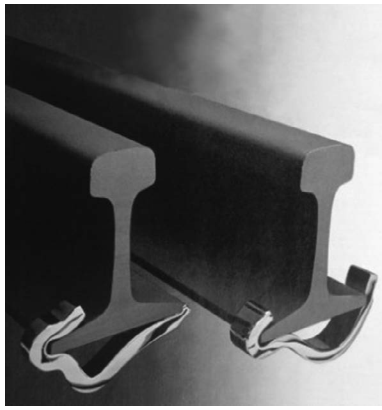


Figure 2.14 Clamp for rail-creep protection.

- The wave motion of the rails caused by the passing trains' wheels, 'pushing' the rails in the direction of movement.
- The weight of the rails, which in the case of longitudinal track gradient, 'pushes' the rails downhill.

For single tracks with bidirectional traffic the rail creep is smaller. For track sections with significant gradient, the rail creep follows the direction of the descending gradient of the track, regardless of the direction of train movement.

The rail creep will result in the gradual accumulation of compressive stresses in some segments of the track and tensile stresses in others, causing a deviation of the RNT from the DRNT and rendering different track segments vulnerable to rail buckling or rail breakage.

The impacts of the rail creep are the following (Lanza Di Scalea, 2012):

- In the case of continuous welded rails increase of the forces exerted due to temperature changes and in the case of jointed rails creation of very large or very small joint gaps.
- Dissimilarities in terms of the degree and extent of the rail creep between the two rails which result in the bending of the sleepers. As a further result, bending moments are developed on the rails.
- Disturbance of the stability of the track's superstructure resulting from the displacement of the rails and sleepers relative to the ballast layer.

The rail creep can be treated using elastic fastenings, good track ballasting and adequate maintenance of the track superstructure.

To prevent rail creep, anti-creep devices are positioned on the track (Figure 2.14). These are U-shaped strips which are placed on the rail foot so as to prevent the longitudinal movement of the rail vis-à-vis the sleeper.

2.4.3 Braking forces: Acceleration forces

The acceleration forces N_{ac} and the braking forces N_{br} are considered as static loads developed during acceleration and braking of trains, respectively (Lichtberger, 2005).

More specifically, during acceleration, longitudinal forces build up on the track due to the static friction between the wheel and the rail. In front of the accelerating rail wheelset, tension is developed while behind the wheelset compression is developed. The magnitude of these longitudinal forces depends on the vertical wheel load as well as on the adhesion coefficient. The acceleration forces can be considered negligible since they do not exceed 5% of the longitudinal forces that are developed due to temperature changes.

When the vehicles are decelerating, compression tendencies appear in front of the first braking wheelset and tensile ones appear behind it, which is precisely the opposite to what happens during acceleration. Another difference in the above two functionalities is that during braking all axles are usually contributing. Braking forces may reach to as much as 15% of the maximum forces due to the variation in temperature. Indicatively, the values of the braking forces for the various vehicles are as follows (Lichtberger, 2005):

- For electric locomotives, they reach values equal to 12%–15% of the axle loads
- For diesel locomotives, they reach values up to 18% of the axle loads
- For 2-axle freight wagons, they reach values up to 25% of the axle loads

The acceleration and braking forces are taken into account in the design of civil engineering structures (bridges, embankments). The braking force is considered as a distributed moving load.

For many railway networks it is considered that the total braking force is equal to 25% of the overall train weight.

2.4.4 Traction forces: Adhesion forces

Problems regarding railway vehicle traction concern the following two cases (Metzler, 1981):

- Determination of the load which a certain type of power vehicle can pull under certain conditions
- Identification of the suitable type of power vehicle which can pull a specific load under certain conditions

These problems can be resolved once the specific features governing the actual transport capacity of the locomotives, and the trains in general, are known.

These features are called ‘traction basic elements’ and are the following:

- Total train resistance (W)
- Power output of the motor or motors (P_t)
- Traction effort developed on the driving wheel treads (F_t)
- Adhesion force generated on the driving wheels’ wheel–rail contact surface (Π)

The fundamental mathematical relation which must apply at all times in order to ensure train movement is the following:

$$\Pi > F_t > W \tag{2.41}$$

In all cases, the traction effort on the treads must be less than the resistance of the couplings.

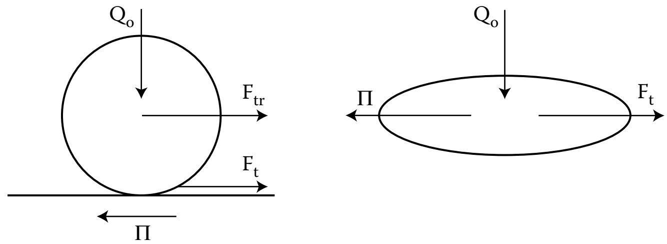


Figure 2.15 Forces appearing during train movement.

The force that is derived as a result of the power output of the motors and acts on the wheel treads (i.e. the contact point between the wheel and the rail, where the motor's torque is converted to force, Riley and Li, 2012), is called traction effort on the treads (F_t).

The force acting on the axles is called traction effort (F_{tr}) (Figure 2.15).

The traction effort on the treads is not constant but alters with the speed of the locomotive and generally reduces as the speed increases. Thus we have the relation

$$P_t = F_t \cdot V \tag{2.42}$$

where

P_t: net (or useful) power

The maximum traction effort on the treads is developed at start up (Figure 2.16). The increase of the speed results initially in the linear reduction of the traction effort on the

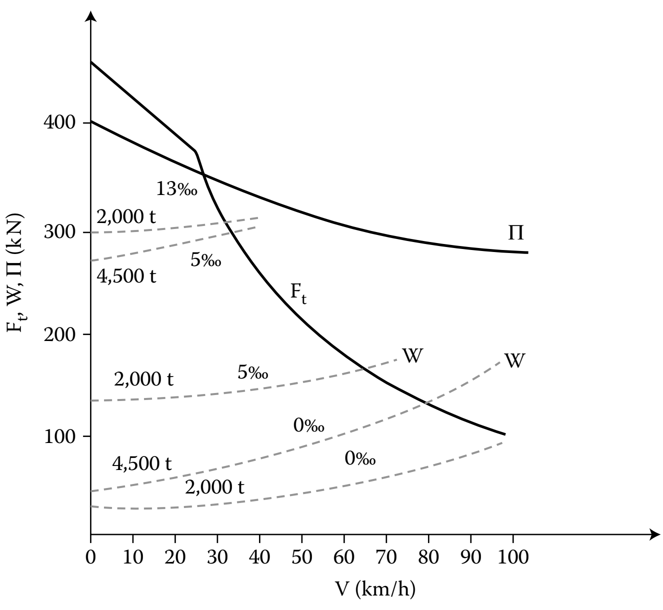


Figure 2.16 Diagram F_t–V, W–V, and Π–V. (Adapted from ABB. 1992, *Traction Vehicle Technic for All Applications*, Information Leaflet, Manheim.)

treads and to its further reduction excessively thereafter. However, the increase of the speed also results in an increase of the train's movement resistance.

It should be noted that an electric locomotive is able to provide for a short time – instantly – traction effort that is greater than the traction effort which it can provide at continuous operation.

The motor's traction effort cannot produce transportation work if it lacks a point of application. On the railway the point of application is to be found on the wheel tread–rail contact surface. The contact surface, the surface that is where the traction effort on the treads (F_t) is applied, must produce a certain resistance, which is the adhesion force (Π) (Figure 2.15).

Adhesion force (Π) is defined as the product

$$\Pi = Q_o \cdot \mu \tag{2.43}$$

where

Q_o: vertical wheel load
μ: adhesion coefficient

The adhesion force depends on many parameters which are classified into three categories namely (Vasic et al., 2003; Cuevas, 2010):

1. Manufacturing features of the vehicle and the track
2. Construction material of the wheel and the rail
3. Environmental conditions

The total train resistance W is equal to the sum of the individual resistances shown in Figure 2.17. Movement resistance depends on the speed and aerodynamic resistance in particular depends on the square of the speed.

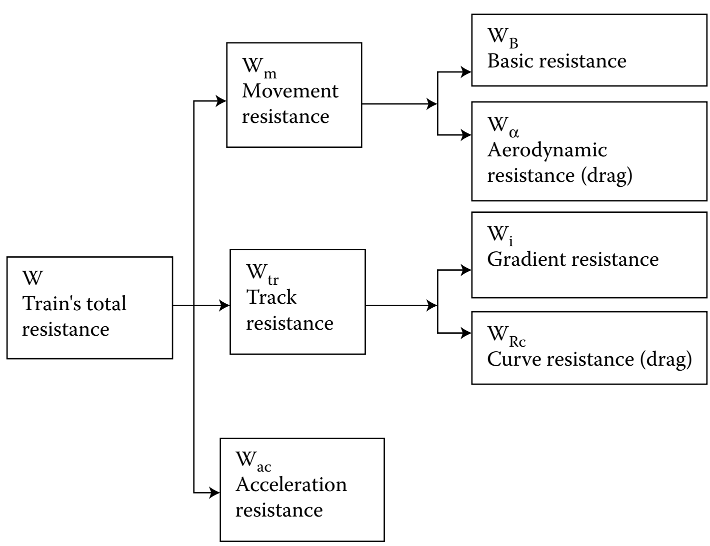


Figure 2.17 Train resistances. (Adapted from Abakoumkin, K. 1986, *Lecture Notes,* NTUA, Athens.)

Figure 2.16 shows indicatively, for a specific type of power vehicle, the variation of F_t, W and Π in relation to speed. This diagram allows for the determination of the maximum speed a train can move at, for a specific type of locomotive at a specific gradient along the track and for a specific power vehicle.

The traction effort on the treads is given by the manufacturer of the rolling stock, while the adhesion force is calculated either experimentally by special trains measuring the friction coefficient or with the help of empirical formulae (Bourachot, 1984; Meccanica della Locomozione, 1998) and the train's total resistance is calculated solely by the competent traction engineer based on his network's data.

According to the diagram illustrated in Figure 2.16, the transport of a load that weighs 4,500 t at a gradient of i = 5‰ can be performed at a speed of V = 38 km/h.

2.4.5 Fishplate forces

For rails that are jointed by fishplates (Figure 2.18) during the movement of trains a force P_f is developed on the joint which can be calculated using the equation

$$P_f = Q_o + 2 \cdot \alpha_f \cdot V_P \cdot \sqrt{k \cdot USM} \tag{2.44}$$

where

Q_o: wheel load (during calculations it is increased by 20% so as to take into consideration the track cant) (kN)

$2\alpha_f$: angle of vertical displacement of the joint (summary of the angles which are formed by the two rails and the horizon) (rad)

V_p: train passage speed (m/s)

k: track vertical stiffness (N/m)

USM: unsprung masses of the vehicle (of one wheelset) (kg)

From the mathematical Equation 2.44 and considering:

- Wheel loads of 11.25 t (freight trains) and 8 t (passenger trains)
- k = 60 MN/m, USM = 1,200 kg
- The angle of vertical displacement in the case of a fishplate link $2\alpha_f$ equals 0.02 rad while in the case of a single welding equals 0.005 rad, respectively (typical values)

Figure 2.18 Rails jointed using fishplates. (Adapted from Les Chatfield from Brighton, 2006, online image available at: https://commons.wikimedia.org/wiki/File:Fishplate_UK_2006.jpg.)

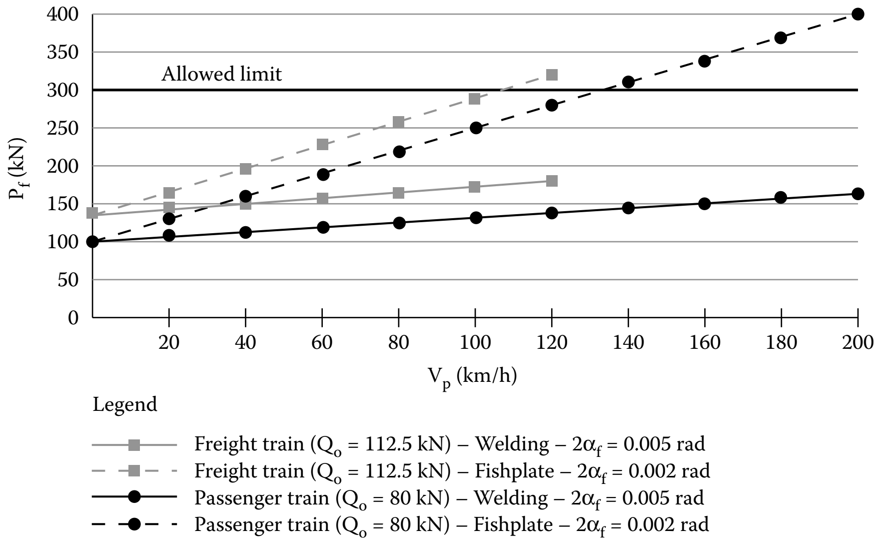


Figure 2.19 Force P_f developed at the rail connection – fishplate joint and welding – passenger and freight trains. (Adapted from Christogiannis, E. 2012, Investigation of the impact of traffic composition on the economic profitability of a railway corridor – Fundamental principles and mathematical simulation for the selection of operational scenario for a railway corridor, PhD thesis, Aristotle University of Thessaloniki, Thessaloniki, Greece.)

the diagram of Figure 2.19 can be derived. Looking at the diagram, the following can be concluded (Bona, 2006):

- The fishplate force P_f increases as the speed increases
- In the case of passenger trains, the force P_f which is developed on the fishplate joint for V_p = 120 km/h is equal to 259 kN, that is, it is about 3 times the wheel load, while as for V_p = 200 km/h, the P_f force reaches a value of 378 kN, that is, about 5 times the wheel load
- In the case of freight trains, the force P_f which is developed on the fishplate joint for V_p = 80 km/h is equal to 244 kN, that is, it is almost double the wheel load, while as for V_p = 120 km/h, the P_f force reaches a value of 303 kN, that is, about 2.5 times the load wheel

From the above and taking into consideration that the maximum value of the force P_f adopted by different networks is around 300 kN, it can be said that for V_p > 120 km/h, the use of fishplate joints is not suitable, while for speeds V_p < 120 km/h the use of fishplate joints may be adopted only for freight-dedicated lines. The technique of continuous welding, especially for lines which are aimed to be used by high-speed trains or heavy haul loads, is the only acceptable technique for the joining of rails.

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Chapter 3

Behaviour of rolling stock on track

3.1 BEHAVIOUR OF A SINGLE RAILWAY WHEELSET

3.1.1 Movement on straight path

The movement of a conventional single (isolated) railway wheelset is known as a sinusoidal or hunting motion (Figure 3.1) and was first studied by Klingel in 1883 (Julien and Rocard, 1935).

Klingel simulated the railway wheelset with a bicone. He assumed that the axle moved at a constant speed V in the direction of the track axis while at the same time, at a random moment in time, it is laterally displaced by 'y' and rotated by an angle 'α'.

Klingel proved that the bicone motion is sinusoidal with a difference of phase of the parameters y and α equal to $\pi/2$ with the following characteristics:

$$\text{Wave amplitude: } y_w \tag{3.1}$$

$$\text{Wavelength: } L_w = 2\pi\sqrt{\frac{r_o \cdot e_o}{\tan \gamma_o}} \tag{3.2}$$

$$\text{Frequency: } f = \frac{V}{2\pi}\sqrt{\frac{\tan \gamma_o}{\gamma_o \cdot e_o}} \tag{3.3}$$

$$\text{Maximum lateral acceleration: } y''_{max} = 4\pi^2 \cdot y_w \cdot \frac{V^2}{L_w^2} \tag{3.4}$$

The reduction of conicity γ_o of the wheels, the increase of the rolling radius r_o and the increase of the length of the railway wheelset $2e_o$ increase the wavelength of the sinusoidal motion and reduce the lateral accelerations of the axle.

Klingel presented a pure kinematic analysis of the phenomenon assuming a harmonious motion without damping and without flange contact. In reality, the motion of a railway wheelset and particularly of a whole vehicle (car body + bogies) is much more complex (Esveld, 2001).

3.1.2 Movement in curves

Let us consider the layout of Figure 1.12. Upon entering the track and trying to achieve equilibrium, the axle is displaced by y_o with regard to the curve's outer face.

Let us calculate the displacement y_o for the above position. The rolling radii of the two wheels will be

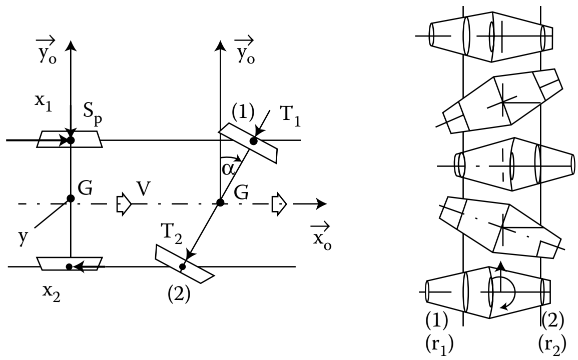


Figure 3.1 Sinusoidal motion of a railway wheelset.

$$r_1 = r_o + \gamma_e y_o \quad \text{and} \quad r_2 = r_o - \gamma_e y_o \tag{3.5}$$

Let us consider S_1 and S_2 as the paths covered by the two wheels during the time interval t, we have

$$S_1 = V_1 \cdot t$$

$$\Rightarrow \frac{S_1}{S_2} = \frac{V_1}{V_2} = \frac{\omega r_1}{\omega r_2} \tag{3.6}$$

$$S_2 = V_2 \cdot t$$

and

$$\frac{S_1}{S_2} = \frac{V_1}{V_2} = \frac{(R_c + e_o)\cdot \xi}{(R_c - e_o)\cdot \xi} = \frac{\omega \cdot (r_o + \gamma_e \cdot y_o)}{\omega \cdot (r_o - \gamma_e \cdot y_o)} \Rightarrow y_o = \frac{e_o \cdot r_o}{\gamma_e \cdot R_e} \tag{3.7}$$

According to the mathematical equation (3.7) the displacement y_o is in reverse proportion to the equivalent conicity γ_e and the radius of curvature R_c. On the contrary, the increase of track gauge and the increase of the wheel diameter lead to an increase of the displacement y_o. For $y_o = \sigma$ we have a contact of the flange with the outer rail, where σ is the flange way clearance.

3.2 BEHAVIOUR OF A WHOLE VEHICLE

3.2.1 Operational and technical characteristics of bogies

3.2.1.1 Object and purposes of bogies

The term bogie sometimes simply denotes a construction that supports the car body without including the wheelsets. However, and this is usually the correct definition, the term refers to the total of 'secondary suspension – bogie frame and primary suspension – wheelsets'.

The ability of the inscription of a railway vehicle in curves depends directly on the length of the vehicle. Initially, railway trailer vehicles were relatively short and their inscription in curved sections of the horizontal alignment was achieved through two or three single wheelsets linked directly to the car body. The evolution of railway as a means of transport went hand in hand with the increase in the vehicles' transport capacity, a fact which dictated the increase in the vehicles' length. Under these circumstances, the inscription of vehicles could no longer be attained using the technique of the single wheelsets.

Using the bogies, the inscription is achieved essentially via the bogies (wheelbase length $2a < 4.0$ m) while the car body follows the movement of the bogies (Figure 3.2).

The bogies must

- Allow the smooth inscription of the wheelsets in curves
- Assist the optimum transfer of loads from the car body to the rails
- Provide stability of the vehicles on straight path (development of high speeds)
- Provide dynamic comfort to passengers in three directions
- Have relatively low construction and maintenance cost

3.2.1.2 Conventional bogies

3.2.1.2.1 Description and operation

Nowadays, 'conventional' or 'classic' bogies are broadly used in trailer and power vehicles. In this technology, the bogies are fitted with wheelsets, the wheels of which are rigidly linked to the axle resulting in the rotation of the wheels and the axle at the same angular velocity (classical wheelsets). The bogie frame is connected to the car body and the wheelset by means of elastic elements and dampers providing the vehicle with two suspension levels namely primary suspension: wheelset–bogie suspension (usually materialised by coil springs

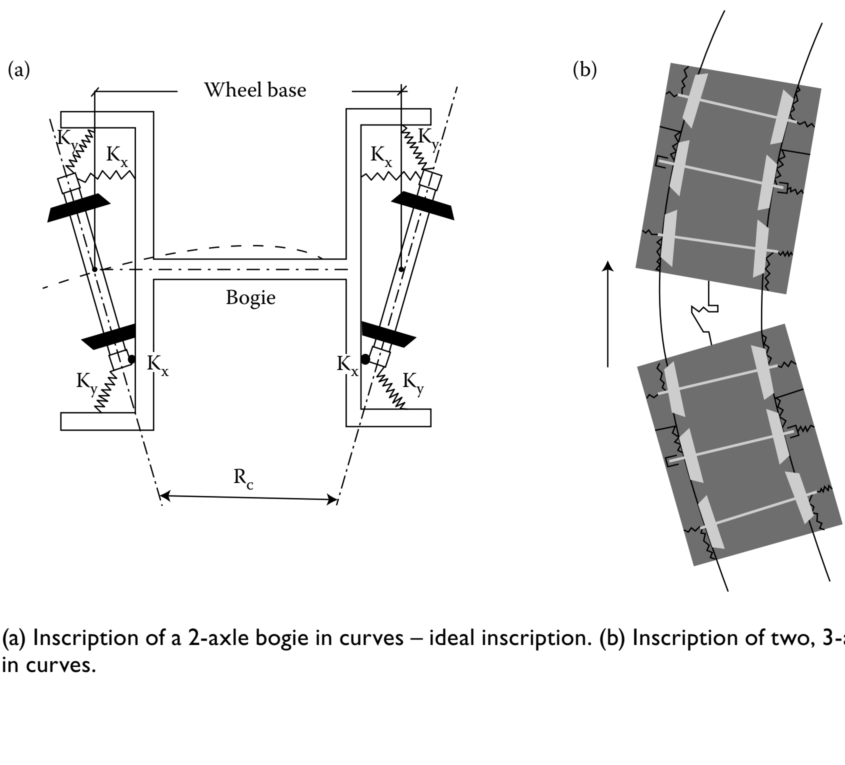


Figure 3.2 (a) Inscription of a 2-axle bogie in curves – ideal inscription. (b) Inscription of two, 3-axle bogies in curves.

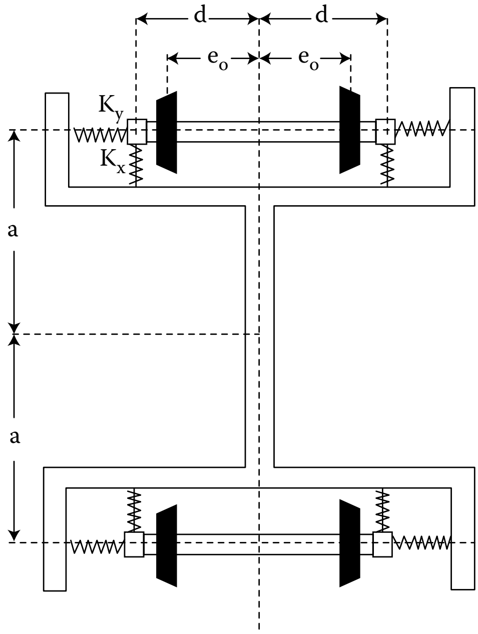


Figure 3.3 Conventional bogie with classical wheelsets.

and dampers or rubber elements [chevrons]) and secondary suspension: bogie–car body suspension (materialised by air suspension or coil springs and dampers) (Figure 3.3).

There are various types of conventional bogies; the choice and design among them depends directly on the functionality of the vehicles to which they will be mounted on and on the geometrical characteristics of the track they will run on.

Figure 3.4 shows in detail all the individual parts which form a conventional power bogie (Schneider Jeumont Rail, no date).

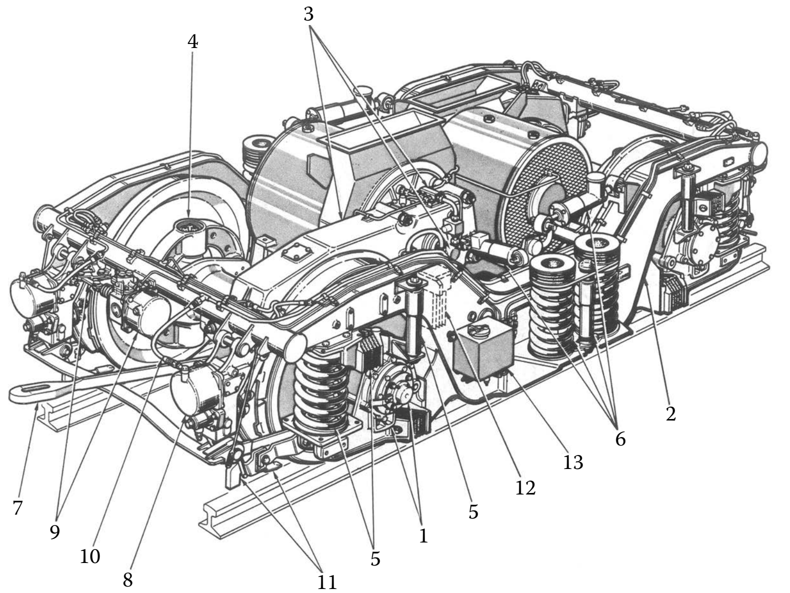
3.2.1.2.2 Design of the bogies

Good construction of the track superstructure does not guarantee on its own a smooth train ride and the achievement of the desired performances; design and construction of the rolling stock is of equivalent importance. Developing a railway bogie from design to commissioning involves the following main stages:

- Conception of the bogie technology and physical explanation of the bogie behaviour
- Theoretical study and modelling of its dynamic behaviour using simulation models
- Design and construction
- Testing
- Commissioning and entering into operation

The geometrical and technical characteristics of the bogies that substantially affect the dynamic behaviour of the vehicles are (Joly, 1983)

- The longitudinal (K_x) and lateral (K_y) stiffness of the primary suspension springs
- The bogie wheelbase (2a) (Figure 3.3)



1. Wheelset–axle box
2. Bogie frame
3. Reducer device
4. Transmitter
5. Primary suspension
6. Secondary suspension
7. Brake lever
8. Air brake
9. Brake pad
10. Brake pipe
11. Sand and stone scattering device
12. Contact brush
13. Wheel flange lubricator

Figure 3.4 Conventional bogie – main parts. (Adapted from Schneider Jeumont Rail. no date, Bogie CL93 à Moteurs Asynchrones, *Catalogue pieces de rechange*, Le Creusot, France.)

- The wheel diameter ($2r_o$)
- The mass of the bogie (M') and of the wheelsets (m)
- The equivalent conicity of the wheels (γ_e)

All the above elements are directly related to the lateral behaviour of the bogies that determine the steady motion of the vehicles on straight paths and the good negotiation of curves as well as

- The car body mass ($\bar{M}$)
- The vertical stiffness ($\bar{K}_z$) of the secondary suspension springs
- The damping coefficients ($\bar{C}_x, \bar{C}_y$, and $\bar{C}_z$) of the secondary suspension dampers

The last three features are directly related to the vertical behaviour of the bogies that characterise the dynamic comfort of the passengers.

Despite the technological advances for the rolling stock and the track equipment, it is not possible to guarantee both high speeds on straight paths and good negotiation of wheelsets in curves.

Indicatively it is noted that (Pyrgidis, 1990)

- The high value of the longitudinal stiffness of the bogie – wheelsets connection (3×10^7 N/m $\geq K_x \geq 10^7$ N/m)
- The small value of the equivalent conicity of the wheels ($\gamma_e < 0.12$)

- And the fixing of devices which restrict the horizontal rotation of the bogie and of the car body (bogie yaw dampers)

allow a conventional railway vehicle to move in complete safety on a straight path of good quality at a speed $V > 350$ km/h. However, for a radius of curvature $R_c < 6{,}000$ m these constructional characteristics relate to

- Wheel slip
- Wheel flange contact with the outer rail

leading to fast wearing out of the wheels and development of guidance forces which in the case of curves with a small radius of curvature ($R_c < 500$ m) can provoke a lateral displacement of the track.

Table 3.1 shows the influence of the constructional and geometrical parameters of the bogies at the vehicle's critical speed V_{cr} (movement along a straight path) as well as on parameters (y, α) which determine the positioning of the wheelsets in curves.

An increase in the critical speed results in an increase in the stability of the vehicle moving on a straight path, hence the possibility of achieving higher speeds. An increase in the lateral displacement 'y' and the yaw angle 'α' of the bogie's front wheelset is equivalent to an increase in creep forces, a likely wheel slip and the appearance of guidance forces (flange contact); and in general an expected poor negotiation of curves (wear on the wheels and the rail, lower speeds, risk of derailment and lateral displacement of the track).

The inability of classical bogies to combine the stable motion of vehicles at high speeds on straight paths with a safe and wear-free negotiation of curves has led to a continuous effort to improve the performance of the wheel–rail system. Within the context of this effort, many improvements have been made regarding the way in which bogies are

Table 3.1 Influence of constructional and geometrical characteristics of bogies

Constructional and geometrical characteristics of bogies	*Movement along a straight path – change of V_{cr}*	*Movement in curves – change of y, α*
Reduction in equivalent conicity γ_e	Increase	Increase
Increase of the wheelbase 2a	Increase	Increase
Increase in the diameter of the wheels $2r_o$	Increase	Increase
Increase in longitudinal stiffness of the bogie–wheelset linkage (K_x)	Increase	Increase
Increase in lateral stiffness of the bogie–wheelsets linkage (K_y)	Increase	Unchanged
Increase in mass of the bogies and the wheelsets (M′, m)	Reduction	Unchanged
Placement of yaw dampers between the bogie and the car body	Increase	Increase

Source: Adapted from Joly, R. 1983, Stabilité Transversale et Confort Vibratoire en Dynamique Ferroviaire, Thèse de Doctorat d'Etat, Université de Paris, Paris; Pyrgidis, C. 1990, Etude de la Stabilité Transversale d'un Véhicule Ferroviaire en Alignement et en courbe – Nouvelles Technologies des Bogies – Etude Comparative, Thèse de Doctorat de l', ENPC, Paris.

Note: Movement along straight path and in curves.

designed and manufactured (new techniques, new elastic connecting materials, lighter bogies, wheelsets, etc.).

One parameter that restricts the performance of conventional bogies is the equivalent conicity γ_e which characterises the wheel wear. This parameter significantly influences the lateral behaviour of the bogies (Pyrgidis and Bousmalis, 2010). The increase in the number of kilometres run by the bogies translates to increased wheel wear and increased initial equivalent conicity; this results in a decrease in the vehicle's critical speed for which it was originally designed. To regain the original performance, the profile of the wheel treads needs to be reshaped at frequent intervals.

3.2.1.3 Bogies with self-steering wheelsets

The behaviour of a bogie in curves is improved when the bogie's wheelsets are placed in a radial way within the curved path. This positioning is not possible with conventional bogies, where generally, bogie–wheelset connections are rigid.

The self-steering (or auto-oriented wheelsets or radial wheelsets or steered bogies) technology allows an almost ideal negotiation of bogies in curves of small radius of curvature ($100 \text{ m} < R_c < 500 \text{ m}$). In this technology, the two classical wheelsets of a conventional bogie are also connected to each other by means of elastic connections of defined stiffness $K_ς$ and K_b (Figure 3.5), where $K_ς$ is the lateral stiffness and K_b the angular stiffness.

The bogie frame–wheelset connection is achieved in different ways (Scheffel, 1974; Scheffel and Tournay, 1980; Joly, 1988; Pyrgidis, 1990).

In the case of conventional bogies, the value of the lateral stiffness K_y between the bogie and the wheelsets depends on the value of the longitudinal stiffness K_x ($K_x = \lambda\ K_y$ where $\lambda < 1$). On the contrary, the technology of self-steering wheelsets allows the manufacturing of springs with independent lateral and angular stiffness values (the angular stiffness K_b plays the same role as the K_x). As the value of the lateral stiffness does not practically influence the negotiation of bogies in curves, it is possible, using this technology, by reducing the angular stiffness and increasing the lateral stiffness to achieve very satisfactory results in small radius of curvature while securing at the same time average values of speed on straight paths (V = 160–220 km/h).

Moreover, this technology seems to provide the wheelsets with a guidance mechanism that tends, in most cases, to position the wheelset inwardly to the curve in a radial way. This technology was first proposed by H. Scheffel in South Africa and it was afterwards

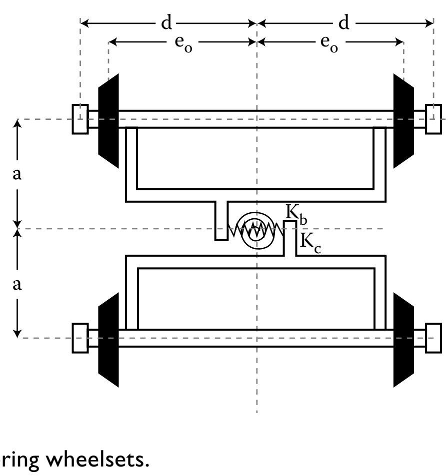


Figure 3.5 Bogie with self-steering wheelsets.

developed in countries with railway networks comprising a significant percentage of curves with small radius of curvature (Scheffel, 1974). This technology is also applied in the bogies of tilting trains (Pyrgidis and Demiridis, 2006).

3.2.1.4 Bogies with independently rotating wheels

In conventional bogies, during a lateral movement of a wheelset on the track, the two wheels of each axle rotate at different rolling radii due to their conical profile. This results in the appearance of longitudinal creep forces of equal value and opposite direction ($X_1 = -X_2$) on each wheel.

To avoid the sinusoidal movement of a wheelset, it is necessary to dispense of the rigid connection of the wheels with the axle in order for the two wheels to be able to rotate at different angular velocities, thus maintaining during their movement the mathematical equation (2.32)

$$\omega_1 \cdot r_1 = \omega_2 \cdot r_2 = V$$

where

ω_1, ω_2: angular velocities of the two wheels
r_1, r_2: rolling radii of the two wheels
V: forward wheelset speed

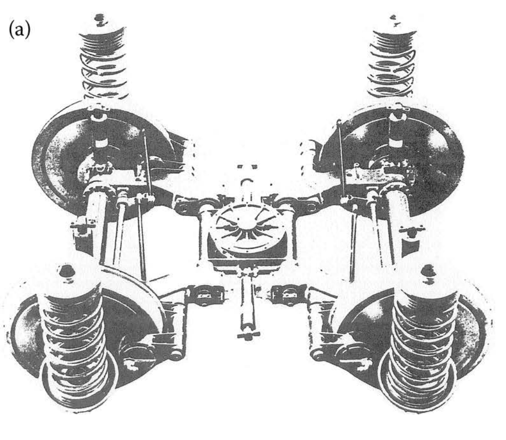
This mathematical equation guarantees wheel rolling without slip and nullification of longitudinal creep forces. This simple reasoning led researchers to develop the technology of bogies with independently rotating wheels (Figure 3.6) (Panagin, 1978; Frullini et al., 1984; Frederich, 1985, 1988; Pyrgidis, 1990).

Using this technology each bogie has four wheels, which rotate at different angular velocities (freely).

Two techniques of implementation of this technology are distinguished

- Bogies with wheelsets (Figure 3.6a)
- Bogies without wheelsets (Figure 3.6b)

(a)



(b)

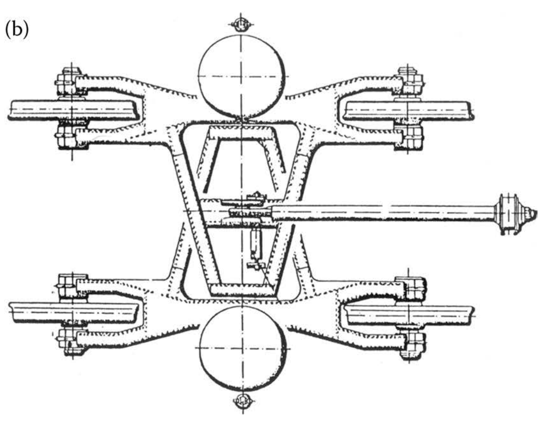


Figure 3.6 Bogies with independently rotating wheels. (a) Bogies with wheelsets and (b) bogies without wheelsets. (Adapted from Frederich, F. 1985, Possibilités inconnues et inutilisées du contact rail-roue, *Rail International*, Brussels, Novembre, 33–40.)

Irrespective of the technical implementation, this technology theoretically allows for the development of very high critical speeds on straight paths. However, the wheelset is very vulnerable to lateral displacement since it can only apply the gravitational force on the track, which is activated at each lateral displacement of the wheelset due to the variable conical wheel profile (Pyrgidis and Panagiotopoulos, 2012). Conversely in curves, wheelsets with wheels, which rotate independently, cannot be placed on the track in a radial position.

It is possible to improve the positioning of the wheelsets in curves should a high value of equivalent conicity be used. This choice does not cause problems on straight paths (since sinusoidal wheelset movement is mitigated), while at the same time, the value of stabilising gravitational forces increases.

The technology of independently rotating wheels is extensively implemented in tramways (Pyrgidis, 2004; Pyrgidis and Panagiotopoulos, 2012).

3.2.1.5 Bogies with creep-controlled wheelsets

In this technology, each bogie bears four wheels rotating at different angular velocities; hence in this case, the mathematical equation (2.32) does not apply.

A magnetic coupling of the two wheels (Figure 3.7) generates a damping torsional torque, the value of which is proportional to the difference of the angular velocities of the two wheels (Geuenich et al., 1985):

$$C_\rho = C_\varphi(\omega_1 - \omega_2)$$

where

C_ρ: damping torsional torque
C_φ: damping coefficient

The torque C_ρ is of the same nature as the one that causes the pair of lateral creep forces, but much smaller.

The bogie wheelset with controlled creeping technology was mainly developed in the United States and Germany. It allows for very high speeds on straight paths without the use of bogie–yaw dampers, thus significantly simplifying the bogie–wheelset connection.

The performance of this technology can be optimised by varying the damping coefficient C_φ as a function of the vehicle forward speed V.

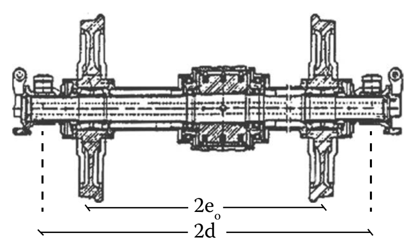


Figure 3.7 Creep-controlled wheelset. (Adapted from Pyrgidis, C. 1990, Etude de la Stabilité Transversale d'un Véhicule Ferroviaire en Alignement et en courbe – Nouvelles Technologies des Bogies – Etude Comparative, Thèse de Doctorat de l' ENPC, Paris; Geuenich, W., Cunther, C. and Leo, R. 1985, Fibre composite bogies has creep controlled wheelsets, *Railway Gazette International*, April, 279–281.)

In spite of its positive impact on vehicle behaviour, the development of this technology was abandoned due to its increased implementation cost.

3.2.1.6 Bogies with wheels with mixed behaviour

These bogies are equipped with a special mechanism that allows them to behave like conventional bogies on straight sections of the track, and to behave like bogies with independently rotating wheels in curves. This technology is applied in tramway vehicles (Pyrgidis, 2004).

3.2.2 Wheel rolling conditions and bogies inscription behaviour in curves

In curves the following wheel rolling and bogie–wheelsets positioning cases may be considered:

1. Rolling of all bogie wheels without flange contact, without slip and without development of creep forces (pure rolling). This case is purely theoretical and is considered to be perfect since due to the absence of forces, no wear is noticed either on the rolling stock or the track.

   In this case, the following mathematical expression applies:

   $$X_{ij} = T_{ij} = 0$$

   and

   $$|y_i| < \sigma \quad (F_{ij} = 0)$$

   where

   X_{ij}, T_{ij}: longitudinal and transversal creep forces exerted on the four wheels of a 2-axle bogie (i = 1,2 front and rear wheelset, respectively and j = 1,2 left and right wheel, respectively, in the direction of movement)

   y_i: lateral displacements of the two wheelsets of a bogie (i = 1,2 front and rear wheelset, respectively)

   σ: flange way clearance

   F_{ij}: guidance forces exerted from the four wheels of a 2-axle bogie to the rails (i = 1, 2 front and rear wheelset, respectively and j = 1, 2 left and right wheel, respectively, in the direction of movement)

2. Rolling of all bogie wheels without flange contact and without slip. The only forces applied on the wheels are the creep forces. This case is considered ideal and may be met under real service conditions where the negative impact on the rolling stock and on the track is considerably minimised (minimal material fatigue and low-level noise emission).

   In this case, the following mathematical expression applies:

   $$\sqrt{X_{ij}^2 + T_{ij}^2} < \mu Q_o \tag{3.8}$$

and

$$|y_i| < \sigma \quad (F_{ij} = 0)$$

where
μ: wheel–rail friction coefficient
Q_o: wheel load.

3. Rolling featuring contact of the front bogie wheelset with the outer rail (via their outer wheel flange) and, accordingly, no contact of the rear bogie wheelset (the rear wheelset may or may not slip) (Figure 3.8).

   This case is frequently encountered in small radii curves. It results in wearing out of the contact wheel and mainly of the outer rail, rolling noise and fatigue of contact materials. This case of inscription is not desirable; however, it is considered acceptable provided that the value of the exerted guidance force F_{11} is not very high and, obviously, it is lower than the limits set by derailment and track lateral shift criteria.

   In this case, the following mathematical expressions pertain:

$$F_{11} \neq 0,\ F_{21},\ F_{22},\ F_{12} = 0, \quad y_1 = +\sigma, \quad y_2 \neq \pm\sigma$$

4. Rolling of both bogie wheelsets in contact with the outer rail (via their outer wheel flanges) (both wheelsets may or may not slip) (Figure 3.9). This case is more adverse than case 3 as two of the four wheels come in contact (via the flange) with the rail and as a result the adverse impact is increased. However, this case may be acceptable provided that derailment and track lateral shift are within the set limits.

   In this case, the following mathematical expressions pertain:

$$y_1 = +\sigma \quad F_{11} \neq 0, \quad F_{12} = 0$$
$$y_2 = +\sigma \quad F_{21} \neq 0, \quad F_{22} = 0$$

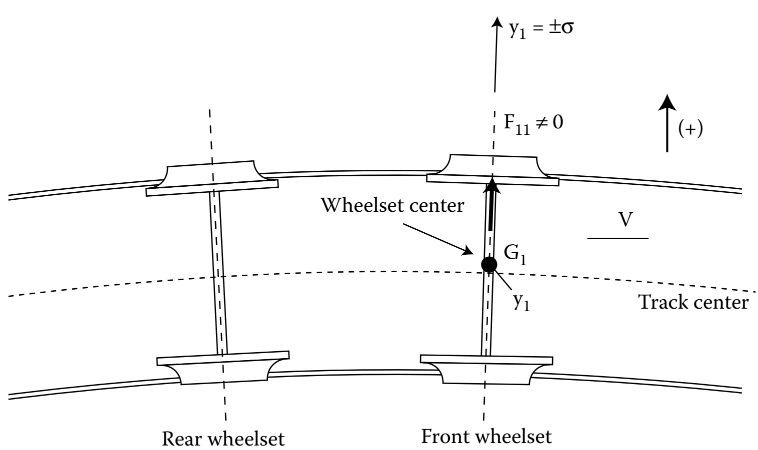


Figure 3.8 Third special rolling condition. Left-wheel flange of front wheelset–outer rail contact – rolling of rear wheelset without wheel flange–rail contact.

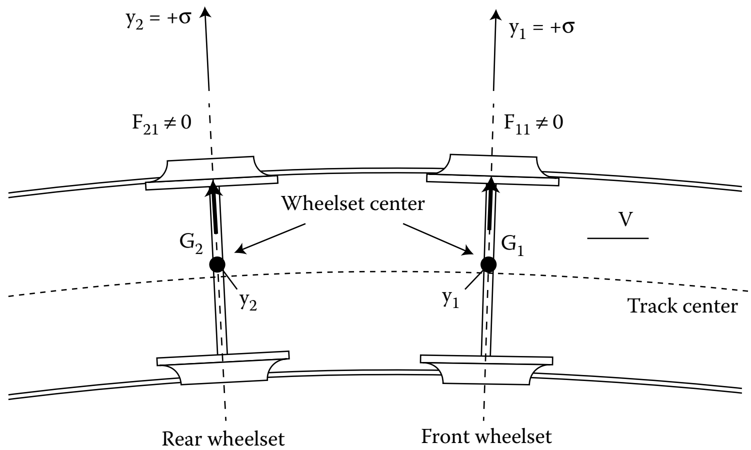


Figure 3.9 Fourth special rolling condition. Left-wheel flange of front and rear wheelset–outer rail contact.

5. Rolling with front bogie wheelset–outer rail contact (via the outer wheel flange) (Figure 3.10). This case, known as 'crabbing', is the most averse and should be avoided as, apart from the adverse impact on the track and rolling stock, the bogie is 'locked' on the track and its displacement is hindered. This case is observed when the primary suspension is particularly rigid and the radius of curvature is small.

In this case, the following mathematical expressions pertain:

$$y_1 = +\sigma \quad F_{11} \neq 0, \quad F_{12} = 0$$
$$y_2 = -\sigma \quad F_{21} = 0, \quad F_{22} \neq 0$$

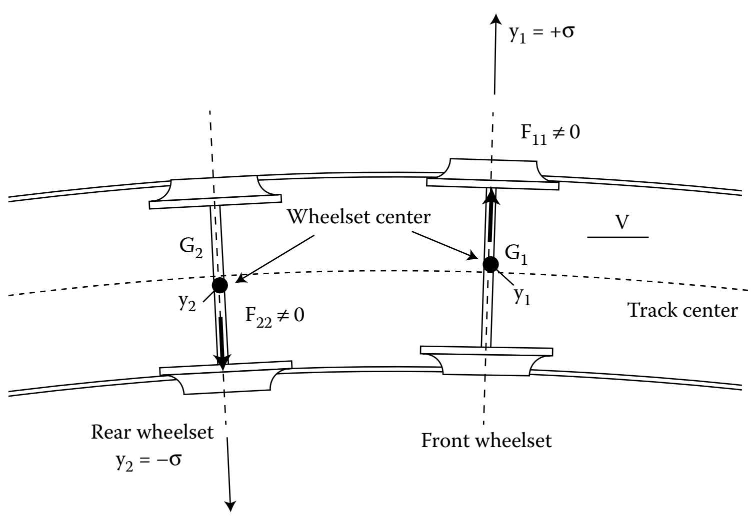


Figure 3.10 Fifth special rolling condition. 'Crabbing rolling': left-wheel flange of front wheelset–outer rail contact. Right-wheel flange of rear wheelset–inner rail contact.

Remark: The wheel slip of a wheelset is always accompanied by contact of the flange of one wheel with the inner rail face (inner or outer rail).

Conditions 3, 4 and 5 are observed during movement along the horizontal curvatures of the track alignment.

3.2.3 Lateral behaviour of a whole vehicle

As mentioned in Section 3.1.1, the movement of a whole vehicle (car body + bogies) is much more complex than that of a single railway wheelset.

Dynamic railway engineering, a division of the applied engineering sector, allows the development of mathematical models simulating the lateral behaviour of a railway vehicle on straight paths and in curves. With the aid of these models it is possible to study the influence of the construction characteristics of the bogies on the 'geometric' behaviour of the vehicle and to determine, for a given speed and for given bogie construction characteristics, the minimum radius of curvature in the horizontal alignment, which ensures acceptable rolling conditions and inscription of the wheelsets in curved sections of the track (avoidance of slipping, absence of guidance forces).

There are many models available in the market. These models are used both in the industry and in academia, and also in research institutes. Whether these models approach reality and to what extent depends on the assumptions and the hypotheses made for their development as well as their mathematical approach. These models are constantly evolving helping to improve traffic safety at all levels, and to achieve a lower vehicle construction cost and a lower cost for the maintenance of track infrastructure and rolling stock.

The simulation models that can be acquired from the market are: SIMPACK, UMLAB, Vampire Pro, Adams/Rail, NUCARS, GENSYS and MEDYNA. All these models take into account the real wheel profile, the track geometric defects and rolling conditions of the wheels. In straight segments these models calculate speeds and accelerations (some models only calculate speeds) and in curves they calculate the applied forces/stresses, the contact surface area and the geometric positioning of the wheelsets on the line (some models only calculate the forces).

Apart from the above models that are available in the market and some other models that are free to use (wheel rail contact calculator) there are some models that have been developed by individual researchers or research groups for their own use, and cannot be found in the market.

A group of such models are described in the literature references (Joly, 1983, 1988; Joly and Pyrgidis, 1990, 1996; Pyrgidis, 1990, 2004; Pyrgidis and Joly, 1993). With these models it is possible to study the following features for five different technologies of bogies (conventional bogies, bogies with self-steering wheelsets, bogies with independently rotating wheels, bogies with creep-controlled wheelsets and bogies with mixed behaviour):

- In the case of tangent track, the lateral vehicle stability and the influence of the main construction features of the bogies on the 'critical' vehicle speed.
- In the case of curved segments of the horizontal alignment, the semi-static lateral vehicle behaviour and the effect of the main features of the bogies on the 'geometric' vehicle behaviour (displacements and yaw angles of wheelsets) and the wheel rolling conditions (calculation of the wheel–rail contact forces, verification of appearance of slipping).

For these models the vehicle moves at a constant speed V and its movement occurs on a railway track without the longitudinal gradient and without geometric tack defects. To study the geometry of the contact the wheel as well as the contact surface of the rail are both simulated by a circle profile (i.e. circle to circle contact) (Joly, 1983).

In the case of a vehicle equipped with conventional bogies the mechanical system consists of the following seven solid bodies that are considered rigid and undeformable: 1 car body, 2 bogies and 4 wheelsets.

At the level of the primary suspension the following were considered per bogie: 4 springs, 1 lateral damper, 1 longitudinal damper and 2 vertical dampers.

At the level of the secondary suspension the following were considered per bogie: 2 springs, 1 lateral damper, 1 longitudinal damper and 2 vertical dampers.

This mechanical system illustrates in fact the French passenger vehicles of the 'Corail' type.

Additionally, the following key assumptions/assumptions are made:

At straight segments:

- The creep forces are expressed on the basis of the linear theory of Kalker and the creep coefficients C_{ij} are considered to be reduced by 33%.
- The rails are not taken into consideration, and as a result the guidance of the wheelsets during the movement is ensured by the combined action of the equivalent conicity of the wheels and the creep forces that are exerted on the wheel–rail contact level. At the same time their lateral stiffness, which is bigger than the stiffness of the vehicle's elastic links, is ignored.

In curves:

- The study of the lateral vehicle behaviour refers to the circular segment of the curve.
- The vertical loads are distributed equally on both wheels of the axles.
- For the calculation of the creep forces the nonlinear theory of Johnson–Vermeulen was adopted (Vermeulen and Johnson, 1964).

3.2.3.1 Vehicles with conventional bogies

Figure 3.11 shows the variation in the vehicle critical speed V_{cr} is given as a function of the longitudinal bogie–wheelsets stiffness K_x for both values of bogie–yaw dampers longitudinal stiffness ($K_o = 3 \times 10^6$ N/m and $K_o = 0$).

There is a zone of values of K_x between 7×10^6 and 1.5×10^7 N/m, where the critical speed reaches its highest values ($V_{cr} = 465–495$ km/h) (Pyrgidis, 1990). This area of K_x values is seen as being the greatest 'safety margin' as regards the stability of vehicles on straight paths for the constructional characteristics of the vehicle illustrated in Figure 3.11.

For values $K_x > 3.5 \times 10^7$ N/m approximately, the critical speed remains roughly equal to $V_{cr} = 450$ km/h. The absence of bogie–yaw dampers reduces the critical speed by about 20%.

The value of $K_x = 8 \times 10^6$ N/m is considered as the optimum value for longitudinal stiffness. On the one hand, this specific value is within the 7×10^6–1.5×10^7 limits and, on the other hand, it is relatively small which facilitates negotiation of curves.

Table 3.2 shows the performances of vehicles with conventional bogies on straight paths and in curves.

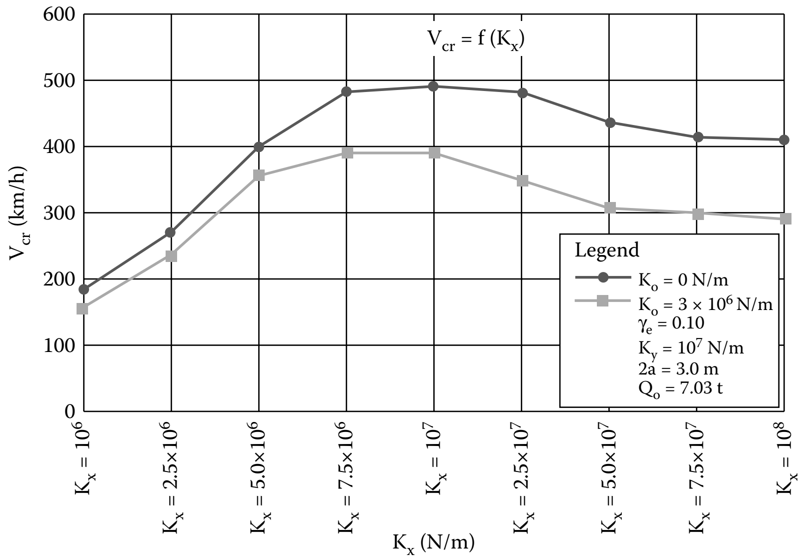


Figure 3.11 Conventional bogies – variation of V_{cr} as a function of K_x. (Adapted from Pyrgidis, C. 1990, Etude de la Stabilité Transversale d'un Véhicule Ferroviaire en Alignement et en courbe – Nouvelles Technologies des Bogies – Etude Comparative, Thèse de Doctorat de l', ENPC, Paris.)

Indicatively, it is noted that

- The high value of the bogie–wheelset longitudinal stiffness (3.5×10^7 N/m $\geq K_x \geq 7 \times 10^6$ N/m)
- The small value of the wheel equivalent conicity ($\gamma_e \leq 0.10$)
- The fixing of devices that limit the horizontal rotation of bogies and car body (bogie–yaw dampers)

allow, in theory, a classical railway vehicle to run on a straight path of good ride quality at speeds V > 600 km/h. However, with such properties, in case of radii $R_c < 4{,}800$ m (for $\gamma_e = 0.10$ and $K_x = 8 \times 10^6$ N/m), we observe (Pyrgidis, 1990)

Table 3.2 Performances of vehicles with conventional bogies – running on straight path and in curves

$K_x = 8 \times 10^6$ N/m	*$2a = 3.0$ m*	*$2e_o = 1.50$ m*	*$\sigma = \pm 10$ mm*
$K_y = 10^7$ N/m	*$2r_o = 0.90$ m*	*$Q_o = 7.03$ t*	*$\gamma_{nc} = 0.02$ g*
Adjustable characteristics	*Straight path (V_{cr})*	*Curves (occurrence of slip)*	*Curves ($R_c = 500$ m) ($K_o = 0$)*
$K_o = 3 \times 10^6$ N/m $\gamma_e = 0.05$	658 km/h	$R_c = 6{,}300$ m (contact)	$F_{11} = 35.1$ kN
$K_o = 3 \times 10^6$ N/m $\gamma_e = 0.10$	482 km/h	$R_c = 4{,}800$ m	$F_{11} = 20.7$ kN
$K_o = 3 \times 10^6$ N/m $\gamma_e = 0.20$	320 km/h	$R_c = 4{,}300$ m	$F_{11} = 0$ (slip)

Source: Adapted from Pyrgidis, C. 1990, Etude de la Stabilité Transversale d'un Véhicule Ferroviaire en Alignement et en courbe – Nouvelles Technologies des Bogies – Etude Comparative, Thèse de Doctorat de l' ENPC, Paris.

- Wheel slip
- Contact of the wheel flange with the outer rail

resulting in an increased wheel wear and in the development of guidance forces.

3.2.3.2 Vehicles with bogies with self-steering wheelsets

If we consider the mechanical system in Figure 3.5 where both wheelsets of a bogie are connected to the bogie using springs of stiffness K_x and K_y and connected to each other using springs of stiffness K_b (angular) KαιK$_s$ (lateral), then the following relations apply (Pyrgidis, 1990):

$$K_{st} = K_s + \frac{d^2 \cdot K_x \cdot K_y}{d^2 \cdot K_x + a^2 \cdot K_y} \tag{3.9}$$

$$K_{bt} = K_b + K_x \cdot d^2 \tag{3.10}$$

where
- K_{st}: overall lateral stiffness of the mechanical system
- K_{bt}: overall longitudinal stiffness of the mechanical system
- 2a: bogie wheelbase

For d = 1.0 m and 2a = 3.0 m relations (3.9) and (3.10) become

$$K_{st} = K_s + \frac{K_x K_y}{K_x + 2.25 K_y} \tag{3.11}$$

$$K_{bt} = K_b + K_x \tag{3.12}$$

and for $K_s = K_b = 0$ relations (3.11) and (3.12) become

$$K_{st} = \frac{K_x K_y}{K_x + 2.25 K_y} \tag{3.13}$$

$$K_{bt} = K_x \tag{3.14}$$

From the above relations, the following may be concluded:

- Stiffnesses K_s and K_b increase the total stiffness (lateral and longitudinal) of the system
- The angular stiffness K_b plays the same role (for d = 1.0 m) as the longitudinal stiffness K_x of a conventional bogie
- The total lateral stiffness of the primary suspension of a conventional bogie depends as much on K_x as on K_y

Table 3.3 shows the performance of bogies with self-steering wheelsets on straight paths and in curves. Compared with conventional bogies, a smaller value of the total longitudinal

Table 3.3 Performances of vehicles with bogies with self-steering wheelset – running on straight path and in curves

$K_b = 10^6$ N/m $K_x = K_y = 10^6$ N/m $K_s = 10^6$ N/m	$2a = 3.0$ m $2r_o = 0.90$ m	$2e_o = 1.50$ m $Q_o = 7.03$ t	$\sigma = \pm 10$ mm $\gamma_{nc} = 0.02$ g	
Adjustable characteristics	*Straight path (V_{cr})*	*Curves (occurrence of slip)*	*Curves ($R_c = 500$ m)*	*Curves ($R_c = 200$ m)*
$K_o = 3 \times 10^6$ N/m $\gamma_e = 0.10$	323.5 km/h	$R_c = 1{,}200$ m	$F_{11} = 15.4$ kN ($K_o = 0$)	$F_{11} = 59.6$ kN ($K_o = 0$)
$K_o = 0$ N/m $\gamma_e = 0.20$	198 km/h	$R_c = 250$ m	$F_{11} = 0$	$F_{11} = 14.8$ kN

Source: Adapted from Pyrgidis, C. 1990, Etude de la Stabilité Transversale d'un Véhicule Ferroviaire en Alignement et en courbe – Nouvelles Technologies des Bogies – Etude Comparative, Thèse de Doctorat de l', ENPC, Paris.

stiffness ($K_{bt} = 2 \times 10^6$ N/m < 8×10^6 N/m) and a smaller value of the total lateral stiffness ($K_{st} = 1.3 \times 10^6$ N/m < 2.62×10^6 N/m) are observed.

Indicatively, it is noted that bogies with self-steering wheelsets make it possible to combine very good negotiation of small and very small radius curves with a fair value of speed on straight paths.

3.2.3.3 Vehicles with independently rotating wheels

Table 3.4 presents the performances of bogies with independently rotating wheels on straight paths and in curves (Pyrgidis, 1990; Pyrgidis and Joly, 1993; Joly and Pyrgidis, 1996).

Indicatively, it is noted that

- This technology allows theoretically, without the fixing of bogie–yaw dampers, the development of very high critical speeds on straight paths while eliminating the hunting of wheelsets (absence of longitudinal creep forces). However, the wheelset is very sensitive to lateral displacements.
- A great value of equivalent conicity facilitates both the negotiation of bogies in curves and the motion on straight paths as it increases the value of the gravitational force that tends to centre the wheelset on the track.
- For wheelbase and wheel diameter values, which are the same as those of high-speed conventional bogies, a wheel slip is observed in comparatively much smaller curvature radii, while the forces exerted in very small radius curves are much smaller.

Table 3.4 Performances of vehicles with bogies with independently rotating wheels – motion on straight path and in curves

$K_x = 10^8$ N/m $K_y = 10^6$ N/m	$2e_o = 1.50$ m $K_o = 0$	$\sigma = \pm 10$ mm $Q_o = 7.03$ t	$\gamma_e = 0.20$ $\gamma_{nc} = 0.02$ g
Straight path (V_{cr})	*Curves (occurrence of slip)*	*Curves ($R_c = 500$ m)*	*Curves ($R_c = 100$ m)*
$V_{cr} \approx \infty$ Vulnerability in lateral displacements	1,400 m	$F_{11} = 1.7$ kN	$F_{11} = 15.7$ kN

Source: Adapted from Pyrgidis, C. 1990, Etude de la Stabilité Transversale d'un Véhicule Ferroviaire en Alignement et en courbe – Nouvelles Technologies des Bogies – Etude Comparative, Thèse de Doctorat de l' ENPC, Paris.

Table 3.5 Performances of examined bogie's technologies in motion on a straight path and in curves

Technology/characteristics of bogies	*Straight path*	*Curves*		
		Occurrence of slip	$R_c = 500$ m, $\overline{K_o} = 0$	$R_c = 200$ m, $\overline{K_o} = 0$
Conventional bogies $K_x = 8 \times 10^6$ N/m $K_y = 10^7$ N/m $\gamma_e = 0.10$ $K_o = 3 \times 10^6$ N/m $2a = 3.0$ m $2r_o = 0.90$ m	482 km/h	$R_c = 4{,}800$ m	$F_{11} = 20.7$ kN	$F_{11} = 65.9$ kN
Bogies with self-steering wheelsets $K_x = K_y = 10^6$ N/m $K_b = 10^6$ N/m $K_s = 10^6$ N/m $\gamma_e = 0.20$ $K_o = 3 \times 10^6$ N/m $2a = 3.0$ m $2r_o = 0.90$ m	226 km/h	$R_c = 250$ m	$F_{11} = 0$ $P_{4w} = 1.11$ kW	$F_{11} = 14.9$ kN
Bogies with independently rotating wheels $K_x = K_y = 10^8$ N/m $\gamma_e = 0.30$ $K_o = 0$ $2a = 3.0$ m $2r_o = 0.90$ m	$V_{cr} = \infty$ Vulnerability in lateral displacements	$R_c = 1{,}400$ m	$F_{11} = 0$ $P_{4w} = 4.3$ kW	$F_{11} = 0$ $P_{4w} = 12.2$ kW

Source: Adapted from Pyrgidis, C. 1990, Etude de la Stabilité Transversale d'un Véhicule Ferroviaire en Alignement et en courbe – Nouvelles Technologies des Bogies – Etude Comparative, Thèse de Doctorat de l', ENPC, Paris.

3.2.3.4 Comparative assessment

In Table 3.5, a comparison of the performances of the three bogie technologies under examination on straight paths and in curves is attempted (where P_{4w} is the power that is consumed at the level of the four wheels of the bogie when rolling occurs without contact between the wheel flange and the inner side of the rail).

3.2.4 Selection of bogie design characteristics based on operational aspects of networks

The selection of bogie design characteristics is directly related to the operational characteristics of the network in which the trains will operate. In this section, technical characteristics for vehicle bogies are suggested considering the operational characteristics of the network. Data are obtained from the application of the mathematical models developed by Joly and Pyrgidis (Joly, 1983, 1988; Joly and Pyrgidis, 1990, 1996; Pyrgidis, 1990, 2004; Pyrgidis and Joly, 1993).

3.2.4.1 High-speed networks

These are characterised by

Track design speed: $V_d \geq 200$ km/h
Alignment layout: Small percentage of curved sections out of total track length. Large and very large curve radii (for $V_d = 200$ km/h and $R_{cmin} = 2{,}000$ m)

The following are proposed for the rolling stock:

Bogies: Conventional
Equivalent conicity: Small (e.g., $\gamma_e = 0.05–0.10$)
Stiffness of the primary suspension: High (e.g., $K_x = 8 \times 10^6$ N/m and $K_y = 10^7$ N/m)
Bogie wheelbase: High (e.g., 2a = 3.0 m)
Wheel diameter: Big (e.g., $2r_o = 0.90$ m)
Bogie and wheelset masses: Small

3.2.4.2 Conventional speed networks

They are characterised by

Track design speed: 140 km/h $\leq V_d <$ 200 km/h
Alignment layout: Mainly medium curve radii (R_c = 500–1,500 m)

Conventional bogies are proposed. The selection of values of the bogie design characteristics depends on the track design speed and the track geometry alignment.

Remark: If it is desired to improve the performance on an existing track (assuming the track superstructure is in very good state) tilting trains may be used.

3.2.4.3 Mountainous networks

Characterised by

Track design speed: $V_d <$ 140 km/h
Alignment layout: Large percentage of curved sections out of the total track length mainly medium and small horizontal alignment radii (R_c = 250–750 m)

The following proposals are made for the rolling stock:

Bogies: With self-steering wheelsets (or conventional wheelsets)
Equivalent conicity: Medium (e.g., $\gamma_e = 0.20$)
Total longitudinal stiffness of primary suspension: Small (e.g., $K_{bt} = 2 \times 10^6$ N/m)
Total lateral stiffness of primary suspension: Small (e.g., $K_{st} = 1.3 \times 10^6$ N/m)

3.2.4.4 Metro networks

These are characterised by

Track design speed: V_d = 90–100 km/h
Alignment layout: Very large percentage of curved sections out of the total track length. Mainly small curve radii (R_c = 150–300 m)

The following options are proposed for the rolling stock:

Bogies: Conventional
Equivalent conicity: High (e.g., $\gamma_e = 0.30$)
Bogie wheelbase: Small (e.g., 2a = 2.00–2.40 m)
Wheel diameter: Small (e.g., $2r_o = 0.70–0.75$ m)
Bogie and wheel masses: Small
Stiffness of primary suspension: Small (e.g., $K_x = 4.10^6$ N/m and $K_y = 10^6$ N/m)

3.2.4.5 Tramway networks

These are characterised by

Track design speed: $V_d = 80–90$ km/h
Alignment layout: Very large percentage of curved sections out of the total track length. Curve radii mainly in the range of $R_c = 25–50$ m

The following options are proposed for the rolling stock:

Bogies: With independently rotating wheels
Equivalent conicity: Very high
Bogie wheelbase: Small (e.g., 2a = 1.80 m)
Wheel diameter: Small (e.g., $2r_o = 0.65$ m)
Bogie and wheelset masses: Small
Longitudinal stiffness in primary suspension: No effect
Lateral stiffness in primary suspension: Small (e.g., $K_y = 10^5–10^6$ N/m)

Apart from the technology of bogies with independently rotating wheels, a mixed system can be used (bogies of the Sirio series tramway). For the correct operation of this technology, a wheel profile with varying conicity is needed to secure small values for γ_e and high values ('smart profile') of γ_e (for the lateral displacements – that is to say during the motion in curves) (Pyrgidis, 2004; Pyrgidis and Panagiotopoulos, 2012).

3.3 DERAILMENT OF RAILWAY VEHICLES

3.3.1 Definition

The term 'derailment' is used to describe the definite loss of contact of at least one vehicle wheel with the rail head rolling surface (Figures 3.12 and 3.13).

The derailment of a railway vehicle may occur as a result of

- Lateral displacement (shift) of the track
- Overturning/tilting of the vehicle
- Wheelclimb

The causes of derailment can be internal (high exerted forces, excessive speed, poor condition and design of rolling stock, poor quality and track layout) or external (incorrect adjustment of switches) (Figure 3.14).

Figure 3.12 Derailment of railway vehicles. (Photo: A. Klonos.)

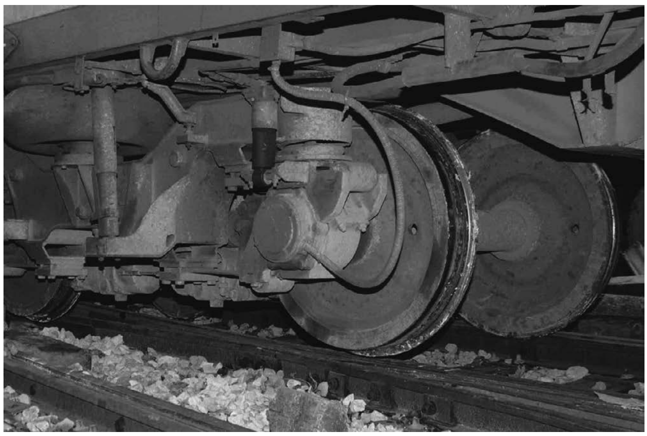


Figure 3.13 Derailment. (Photo: A. Klonos.)

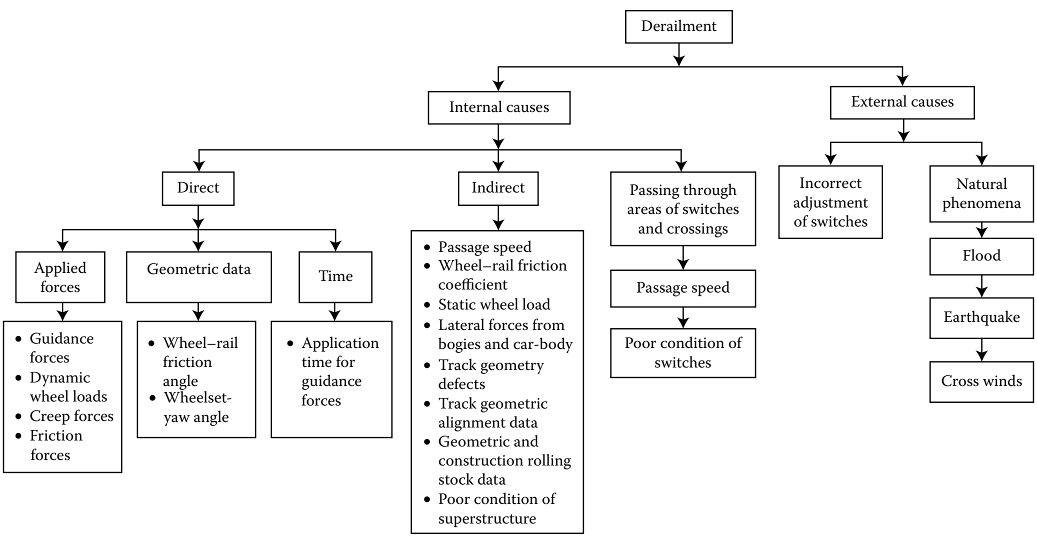


Figure 3.14 Causes of derailment.

3.3.2 Derailment through displacement of track

In this case, the track panel (rails + sleepers) of a track segment is displaced due to the effect of significant lateral forces, resulting in the derailment of one or more of the train's vehicles.

Derailment through displacement of the track occurs when

$$\Sigma Y > H_R \tag{3.15}$$

where

ΣY: total lateral force, which is transferred from the vehicle to the rail

H_R: lateral track resistance

This type of derailment is solely due to internal causes and is the most common type of derailment.

3.3.3 Derailment as a result of vehicle overturning

During movement or the immobilisation of a railway vehicle on curved sections of the horizontal alignment, the vehicle may overturn under certain conditions.

Overturning may occur toward the outside or the inside of the curve.

In the first case (Figure 3.15a), the following reasons may pertain (Esveld, 2001):

- Significant deficiency in relation to the passage speed V_p and the radius of curvature, which translates to an increased value of the lateral residual centrifugal force F_{nc}
- Crosswind force H_w directed toward the outside of the curve
- Unequal load distribution on two wheels with lower loading on the inside wheel ($Q_1 > Q_2$)

All the above reasons result in the development of moments, which tend to overturn the vehicle toward the outer rail.

In the second case (Figure 3.15b) the following reasons may pertain, respectively,

- Crosswind force H_w directed toward the inside of the curve
- Immobilisation of vehicles ($V_p = 0$) on a curved track section with a high cant U
- Small axle load
- Displacement of load toward the inside wheels

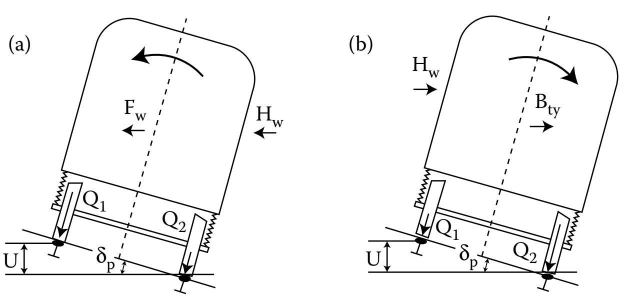


Figure 3.15 Vehicle overturning mechanism in curves (Adapted from Esveld, C. 2001, *Modern Railway Track*, 2nd edition, MRT-Productions, West Germany.): (a) Toward the outside of the curve and (b) Toward the inside of the curve.

Under these circumstances moments which tend to overturn the vehicle toward the inner rail develop resulting ultimately in derailment.

3.3.4 Derailment with wheel climb

3.3.4.1 Description of the phenomenon

Assuming the case of rolling in Figures 3.13 and 3.16 (Alias, 1977), for the wheel, which is close to a derailment (wheel 1) and the vertical load Q_1 and the force Y_1 (total force exerted via the wheel flange on the rail) are applied on the point of contact I_1. The reaction R_1 of the rail may be analysed in two components:

- A force N_1 that is perpendicular to the level of wheel–rail contact xOy
- The lateral creep force T_1 which acts on the level of wheel–rail contact, and is directed upwards, having a value equal to

$$T_1 = C_{22}\alpha_{at} = N_1 \tan \beta_1 \tag{3.16}$$

where
α_{at}: angle of attack (wheelset yaw angle when flange contact occurs) (Figure 3.16)
C_{22}: lateral creep coefficient

In the case where wheel 1 slips, the force T_1 is equal to Coulomb's friction force.

In practice, derailment through wheel climb occurs when the projection of the combination of all forces applied on the axis yy (derailment force axis) are directed upwards and their application time is long enough for the wheel to climb over the rail.

3.3.4.2 Derailment criteria

For testing against derailment various criteria are being used such as the Nadal criterion, the Chartet criterion, etc. (Alias, 1977). Some of these criteria take into account the yaw angle of the wheelset that is under a derailment while others do not. Figure 3.17 in combination with the mathematical expression (3.17) illustrate the Nadal criterion:

$$Y_1 < Q_1 \frac{\varepsilon\varphi\beta - \mu}{1 + \mu\varepsilon\varphi\beta} \tag{3.17}$$

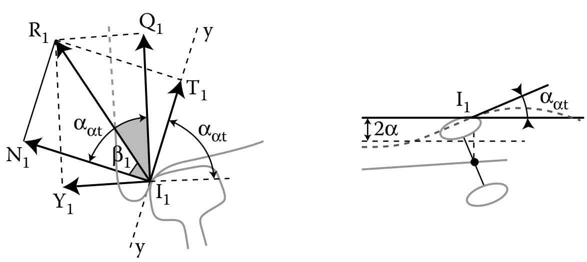


Figure 3.16 Derailment through wheel climb. (Adapted from Alias, J. 1977, *La Voie Ferrée*, Eyrolles, Paris.)

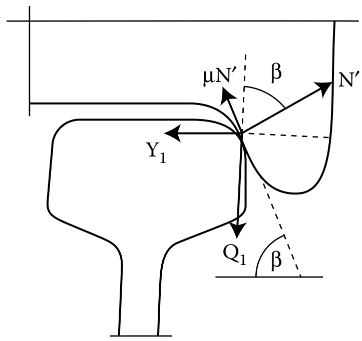


Figure 3.17 Nadal's derailment criterion. (Adapted from Alias, J. 1977, *La Voie Ferrée*, Eyrolles, Paris.)

where

Y_1: the total transversal force exerted on the rail via the wheel flange of the derailing wheel (wheel 1)
μ: wheel–rail friction coefficient
β: wheel–rail contact flange angle
Q_1: static load of wheel 1

For $\beta = 70°$, $\mu = 0.25$ (dry rail), the mathematical equation (3.17) results $Y_1/Q_1 = 1.5$ while as for $\mu = 0.12$ (wet rail) it results $Y_1/Q_1 = 2.0$.

3.3.4.3 Factors affecting derailment

In most occasions, the derailment of a vehicle takes place following a lateral displacement of the track.

Derailment by rail climbing can occur only when there is a significant unloading of the derailed wheel with simultaneous loading of the non-derailed wheel. This phenomenon can be observed in the case of movement at low speeds in curves with a small radius of curvature and high values of cant and twist.

Derailment by rail climbing also occurs through external causes, that is, poor operation and adjustment of switches, etc.

It should be noted that most derailments occur in areas of switches and crossings.

The risk of derailment due to rail climb increases when there is

- An increase of the value of the Y_1 force
- An increase in the application time of Y_1 force
- An increase in the value of the wheel–rail friction coefficient μ
- A decrease in the value of the wheel–rail contact angle β
- A decrease in the value of the vertical load on the derailed wheel with a simultaneous increase of the vertical load of the non-derailed wheel

Rail climb by the wheel does not occur instantaneously. A certain amount of time is required, and thus the derailing wheel covers some distance on the track, usually some metres. This distance is called 'flange-climbing distance' and is defined as the distance covered from the moment when the total value of the guidance force is applied until the moment on which the wheel–rail contact flange angle reaches 26.6° (Dos Sandos et al., 2010). The shorter this distance is, the faster the derailment will become apparent.

The influence of various wheel parameters on the above distance was examined with the aid of simulation modelling. The results have shown that the flange-climbing distance increases (and thus the appearance of derailment slows down) when (Dos Sandos et al., 2010)

- The angle of attack is smaller.
- The height of the wheel flange is increased. When the wheel is worn out the distance required for rail climbing increases. The positive effect of a high flange is significantly limited when the yaw angle is large.
- The value of q_r increases (see Figure 1.8).

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Chapter 4

Tramway

4.1 DEFINITION AND DESCRIPTION OF THE SYSTEM

The modern tramway is a steel wheel electric train, running almost exclusively at grade along urban or suburban roads. It either shares the same infrastructure as the rest of the road traffic, or it moves on a specially built corridor or, finally, on a segregated (protected) lane, placed at one side, at two opposite sides or in the middle of the roadway.

It serves distances usually in the range of 5–20 km and it may be integrated into horizontal alignment radii as tight as 20–25 m. It is characterised by commercial speeds in the range of 15–25 km/h, and it is able to carry about 15,000 passengers/h/per direction (Brand and Preston, 2005). It commonly uses two, one-way traffic lines (double track), which are constructed either with grooved rails embedded in the pavement, or with conventional flat bottom rails.

As far as technology and operation are concerned, the modern tramway is a newer version of the conventional tram (streetcar) which monopolised the urban public transport of most cities in Europe and the United States in the early decades of the last century (Figures 4.1 and 4.2).

4.2 CLASSIFICATION OF TRAMWAY SYSTEMS

Figure 4.3 summarises the main categories in which tramway systems can be classified based on their operational and constructional characteristics.

4.2.1 Physical characteristics of the corridor

Tramway corridors may be classified into five different categories (classes: E, D, C, B, A) (Bieber, 1986).

a. *Common corridor (class E):* In this case, railway vehicles are mixed with road vehicles and pedestrians (Figure 4.4).
   In order not to hinder the movement of road vehicles, a tram runs on special rails (grooved rails), which are properly embedded in the pavement. The implementation cost of this corridor is relatively low but the train's commercial speed remains low, similar to the commercial speed of urban buses (12–15 km/h) (Bieber, 1986). Furthermore, priority at traffic lights in relation to road transport cannot be given to the tram.
b. *Exclusive separated corridor' (class D):* In this case, grooved rails are also used, but they are separated from the general traffic by means of horizontal lining or obstacles accessible to pedestrians (Figure 4.5).

Figure 4.1 Streetcar. (From Collection CERTU 1999, *Nouveaux Systèmes de Transports Guidés Urbains*, Paris, March 1999.)

The tramway is, theoretically, separated from the rest of the traffic, except at level crossings. Separation of a corridor increases the commercial speed of the trains (16–22.5 km/h) (Bieber, 1986; Pyrgidis and Chatziparaskeva, 2012).

At road intersections, level crossings are maintained, however, at these locations, priority at traffic lights can be given to the tram.

c. *'Exclusive tram corridor' (class C):* The existing road is used exclusively for the movement of the tram while the remaining road width is pedestrianised. The solution is applicable for narrow streets or when it is deliberately sought for sole use of the road by the tramway traffic only (for instance, in the case of commercial or historical city centres).

Grooved rails, which are embedded in the pavement, are used for the construction of the corridor, while the segregation of the tram corridor from the pedestrian area is usually achieved with the use of horizontal signalling (Figure 4.6).

Figure 4.2 Modern Tramway, Zagreb, Croatia. (Photo: A. Klonos.)

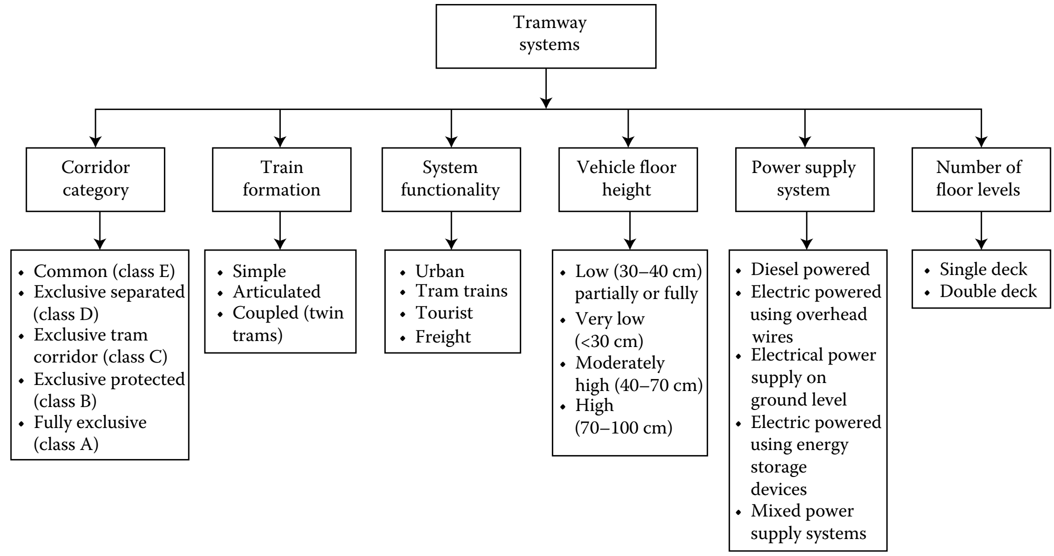

Figure 4.3 Classification of tramway systems based on their different constructional and operational characteristics.

These three solutions deliver a relatively small improvement in the quality of service in comparison with urban buses or trolleys moving on an exclusive lane, while at the same time allowing the implementation of a tram line with a relatively low construction cost. On the contrary, the environmental impact of such an intervention is very positive.

d. '*Exclusive protected corridor' (class B):* In this case, the tramway is completely separated from the circulation of road vehicles and pedestrians. The separation can be achieved by artificial or natural means (trees, plants, railings, walls, etc.), and pedestrian crossings are placed at specific intervals, depending on pedestrian flows (Figure 4.7).

Figure 4.4 Tram operation on 'common corridor' (class E), Oslo, Norway. (Photo: A. Klonos.)

Figure 4.5 Tram movement on 'exclusive separated corridor' (class D), Amsterdam, Holland. (Photo: A. Klonos.)

Railway vehicles usually move on rails that are similar to those of classic rail tracks (Vignoles type) but are of lighter construction. However, there is always the possibility of embedding them, as in the example of Figure 4.7. At intersections with roads, level crossings are maintained, but at these locations priority at traffic signals can be given to the tram.

The movement of trains occurs with no particular problems, and despite the presence of level pedestrian crossings it is possible to achieve commercial speeds of 20–25 km/h (Bieber, 1986).

e. *'Fully exclusive corridor' (class A):* In this case, tramway vehicles move as in the previous case (class B) on flat bottom rails, at grade, or underground, or elevated section (Figure 4.8).

Figure 4.6 Exclusive tram corridor (class C), Zurich, Switzerland. (Photo: A. Klonos.)

Figure 4.7 Tram movement on 'exclusive protected corridor' (class B), Athens, Greece. (Photo: A. Klonos.)

The level pedestrian crossings and intersections with roads are removed so that the commercial speed reaches 30 km/h (Bieber, 1986). This type of corridor is also used for the light metro and suburban rail.

The two latter solutions provide a better quality of service, but are comparatively far more expensive, especially when it is required that underground or elevated track sections are built.

4.2.2 Functional/operational criteria

On the basis of functional/operational aspects and, more specifically, based on the nature and extend of services they provide, tramway systems may be divided into four categories:

Figure 4.8 Tram movement on 'fully exclusive corridor' (class A), Paris, France. (Photo: A. Klonos.)

a. *Urban tramways:* Serve passenger movements for relatively short distances (S = 5–20 km) within an urban area; they move at low commercial speeds (V_c = 15–25 km/h).

b. *Long-distance tramways (tram-trains):* This technique was first applied in Germany for the Regio Citadis train at Karlsruhe (1992, 1997). Tram-trains usually serve trips that are 15–50 km long, connecting city centres to suburban and periurban areas. The maximum running speed that can be developed is V_{max} = 80–120 km/h, and the commercial speed is around V_c = 60 km/h.

   Tram-trains are operated on infrastructure which is used not only by trams but also by other categories of railway systems (suburban, commutter and regional passenger trains, freight trains) (Figure 4.9). The vehicles are equipped, for example, with two traction systems (dual mode vehicles, diesel/750 V DC) or with dual-current systems (dual-voltage vehicles, e.g., 15 kV AC and 750 V DC), whilst the vehicle design is such that it allows trains to operate on platforms of different heights. Thus, the need to change mode is eliminated, accessibility is improved and travel times are reduced.

   The length of tram – trains range between 26.50 and 37 m, the width ranges between 2.40 and 2.65 m, the height of the floor ranges between 350 and 450 mm with folding stairs and, finally, the bogies can negociate curves of horizontal alignment radii up to R_c = 20–30 m. The average distance between stops ranges from 500 m up to 5 km.

c. *Tourist tramways or cultural heritage trams:* These systems serve tourist and recreational needs. They have a short connection length and move at low commercial speeds.

d. *Transport of freight:* Since the beginning of the twenty-first century, urban tramway systems have been used for freight. The incentive is to reduce air pollution, traffic congestion and the wear and tear of city centre traffic infrastructure. Urban trams that are able to carry goods are those at Dresden (Figure 4.10), Cologne (Germany) and Zurich (Switzerland). In Amsterdam, such trams were pilot tested, but ultimately not commissioned for revenue service.

Figure 4.9 Regio Citadis, Hague, The Netherlands. (Photo: A. Klonos.)

Figure 4.10 Freight tram in Dresden, Germany. (Adapted from http://www.flickr.com/people/77501394@N00 kaffeeeinstein, 2008.)

4.2.3 Floor height of the vehicles

Depending on the distance between the floor of the vehicle and the top of the rail, tramway systems are divided into low floor, very low floor, moderately high floor and high floor.

4.2.3.1 Low floor

The height between the rail running table and the floor of the vehicle is 30–40 cm (commonly 35 cm) resulting in passenger access to vehicles without any steps (Figures 4.11 and 4.12).

Figure 4.11 Wheelchair access at low floor tram. (Adapted from Lasart75, 2010, available online at http://en.wikipedia.org/wiki/Low-floor_tram (accessed 7 August 2015).)

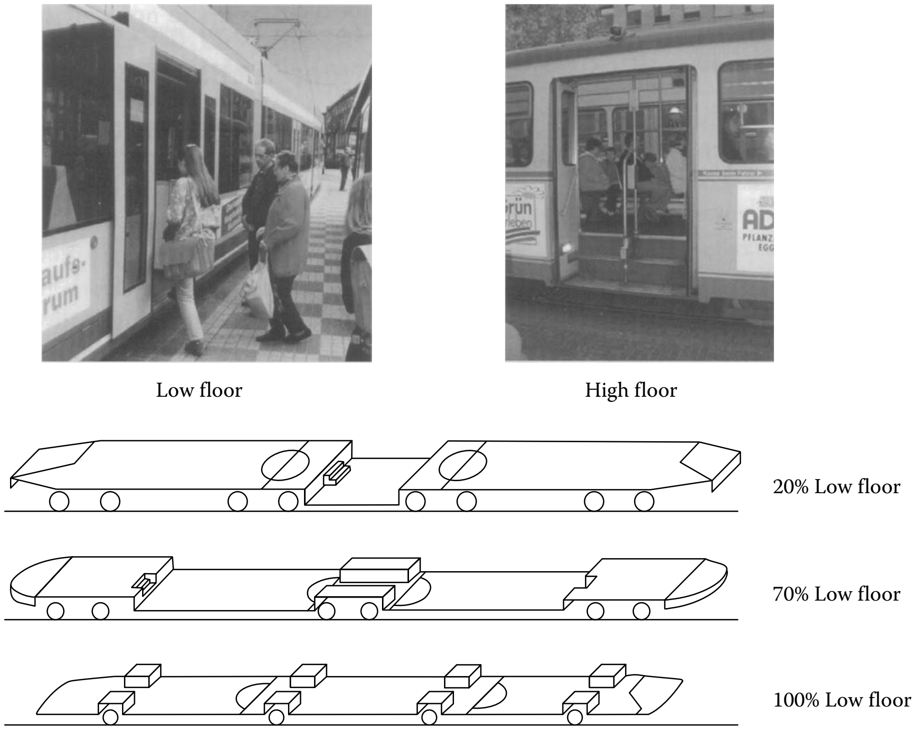

Figure 4.12 Low and high floor tram. (Adapted from Hass-Klau, C. et al. 2003, Bus and Light Rail: Making the Right Choice, ETP, Brighton.)

This design improves the tram's accessibility for the public, and particularly for people with disabilities, and allows for the construction of larger windows. However, it also generates some problems such as

- Difficulty in the installation and maintenance of electrical equipment.
- Discontinuity at floor level where carriages connect to each other.
- Positioning of the equipment at the edges of the car body, thereby increasing the length of the vehicle by 10%–20%.

The minimum height of the platforms is considered to be 0.25 m. If low-floor vehicles are selected, the platforms' height can be reduced or even eliminated.

4.2.3.2 *Very low floor*

The height between the rail running table and the floor of the vehicle is <30 cm. A typical value for that height is 180 mm.

4.2.3.3 *Moderately high floor*

The height between the rail running table and the floor of the vehicle is 40–70 cm. For such trams there are stairs in order for the passengers to access the vehicle floor.

4.2.3.4 High floor

The height between the rail running table and the floor of the vehicle is 70–100 cm. For these trams, passengers' access to the vehicle floor is achieved by means of steps (Figure 4.12).

Depending on the percentage of their length which is low floor, trams are distinguished (Figure 4.12) in the following categories:

- Totally (l00%) low floor (low floor throughout the length of the vehicle)
- Partially low floor (e.g., for 70% of the vehicle length)

The technology of partially low-floor trams is old. On the contrary, the technique of low-floor trams is relatively new and is now an established current state-of-the-art for the urban tram. The first vehicle of this type was manufactured as a laboratory prototype by Socimi Company in Milan in 1989. In 1990, the German company M.A.N. GHH presented the prototype of the first totally low-floor vehicle (350 mm) in Bremen.

4.2.4 Power supply system

On the basis of the power supply system that is used for their movement, modern tramway systems are divided into the following categories:

- Diesel-powered
- Electrically powered via overhead wires
  - Trolley type
  - Overhead catenary systems (single or dual-voltage, πχ. 15 kV AC and 750 V DC)
- Electrical power supply at ground level
  - Conventional system of third rail
  - Conventional system of fourth and fifth rails
  - APS (Alimentation Par Sol)
  - Tramwave
  - Primove (the transfer of energy to the vehicle is inductive [contactless])
- Electrical power supply, via energy storage devices (supercapacitors charged at stops via overhead wires)
- Mixed supply system
  - Diesel-powered and electrically powered via overhead wires (dual mode)
  - Electrically powered via overhead wires and power supply at ground level
  - Electrically powered via overhead wires and energy storage devices (supercapacitors, batteries)
  - Power supply at ground level, and energy storage devices (supercapacitors, batteries)

The vast majority of tramway systems are powered via the catenary system, however, the techniques that do not use overhead wires have largely developed over the last few years. These schemes are considered to be cutting-edge technologies for rail, and are examined in detail in Chapter 20.

4.2.5 Other classifications

On the basis of the formation of trains, trams may be divided into simple, articulated (Figure 4.13) and coupled (articulated or simple).

Figure 4.13 Double articulated tram, Hague, The Netherlands. (Photo: A. Klonos.)

Basing on the number of levels of their floors, vehicles are distinguished as

- Single deck
- Double deck

Double-decker trams were used extensively in Great Britain until 1950 when they were dismantled. Today, this type of tram is still in operation in Alexandria, Blackpool and Hong Kong.

Basing on the bogie's technology, modern trams are divided into those using bogies with independently rotating wheels and those using bogies of mixed behaviour (see paragraph 3.2.1.6).

Finally, based on their construction history, urban tram systems can be divided into the following three categories:

- Category 1: This category includes new systems that were built after 1980.
- Category 2: This category includes systems that were built many years ago. These systems were taken out of circulation and their tracks were dismantled. However, new infrastructure has recently been built for these systems and they were reopened.
- Category 3: This category includes systems that were built many years ago, and were modernised and upgraded.

4.3 CONSTRUCTIONAL AND OPERATIONAL CHARACTERISTICS OF THE SYSTEM

The main characteristics of tramway systems were presented in Table 1.6. Table 4.1 provides additional data on the characteristics of tramway systems. In addition, the following should also be mentioned.

4.3.1 Data related to track alignment and track superstructure

The cant of the outer rail at curved sections of the horizontal alignment is only deployed for the case of a fully exclusive tramway corridor (class A). For all other tramway corridor categories, the cant is avoided.

Table 4.1 Features and characteristic values related to tramway systems

Minimum horizontal alignment curvature radius	R_c = 20–25 m, preferred value $R_c \geq 30$ m R_c = 15–18 m at shunting tracks
Types of track integration	• A single track per direction at the two opposite sides of the roadway • A double track on one side of the roadway • Central alignment (double track)
Types of stops integration	• Stop with centre (island) platform • Stop with laterally staggered platforms • Stop with laterally opposed platforms
Types of power supply system (catenary overhead system integration)	• Central mast and opposite cantilevers • Lateral mast and double-track cantilevers • Catenary connected to laterally opposed masts • Catenary connected to building facades • Mixed catenary connection (on one side to lateral masts and on the other to building facades)
Vehicle length	Simple: 8–18 m, Articulated: 18–30 m, Multiarticulated: 25–45 m
Vehicle width	2.20–2.65 m (normal track gauge)
Commercial speed per tramway corridor category (without tram priority at level intersections)[a]	Class (E): V_c = 12–15 km/h Class (D): V_c = 16–18 km/h Class (C): V_c = 18–20 km/h Class (B): V_c = 20 km/h Class (A): V_c = 30 km/h
Impact of tram priority at signals on the commercial speed of the trams	Increase of commercial speeds by 15%–25% for corridor classes (D) (V_{cmax} = 22.5 km/h) and (B) (V_{cmax} = 25 km/h)

Source: Adapted from Pyrgidis, C. 1997, Light rail transit: Operational, rolling stock and design characteristics, *Rail Engineering International*, No 1, 1997, Netherlands, pp. 4–7.

[a] Commercial speeds for each tramway corridor category result empirically considering an average distance between intermediate stops equal to 500 m, a halt time of 20 sec and no priority at signals (Bieber, 1986).

Generally its use should be avoided in the vertical alignment radii R_v that are smaller than 150–200 m.

Basing on the tramway corridor, the track may be laid in two distinct ways:

- For the case of a 'fully exclusive corridor' (class A) and in some cases, for 'exlusive protected corridor' (Class B), the rails used are similar to those of classic railway tracks (Vignoles/flat-bottom type), but of lighter construction (46 kg/m or 50 kg/m).

  For the construction of the superstructure, the excavation is usually 70 cm deep in order to ensure a unified sub-base. The track is ballasted or made on concrete (slab track). For intersections with road arteries, a special construction is foreseen.
- In the case of corridors of classes E, D, C and in many cases of B, in order not to prevent the movement of other vehicles and the crossing of pedestrians, special types of rails which are embedded in the pavement (grooved rails, Figure 4.14) are used. In both cases, in order to reduce the vibrations, resilient fastenings are used, elastic pads are placed under the sleepers and antivibration layers may be employed under the ballast.

In special cases, high-damping, mass-spring systems (floating slab tracks) may be used. These systems (see Chapters 5 and 19) allow an even greater reduction of vibrations, however, they increase the implementation cost significantly.

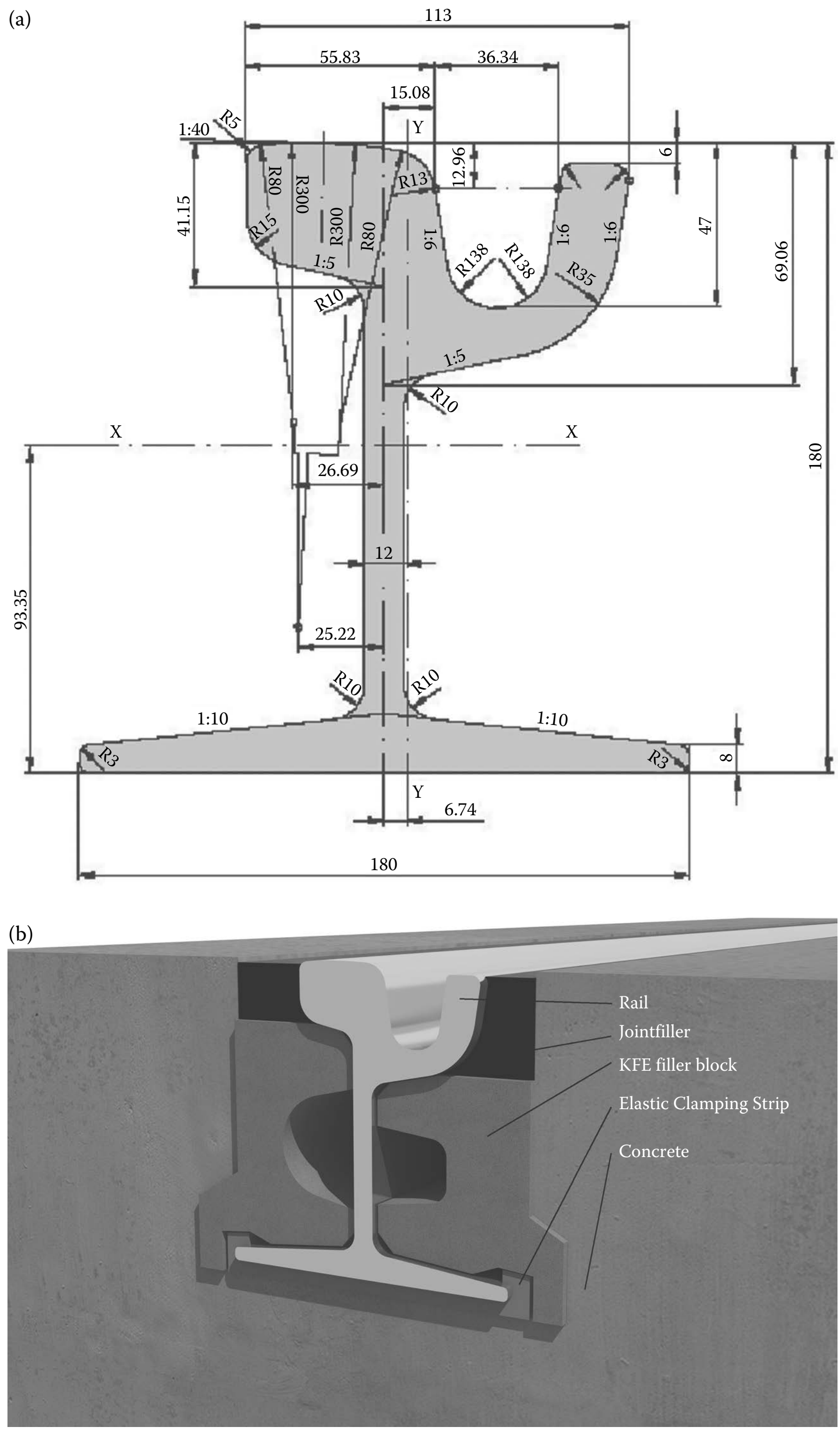

Figure 4.14 (a) Grooved rail Ri60N, cross section and (b) embedded grooved rail, sound damping system (From edilon)(sedra, 2015.)

4.3.2 Rolling stock data

Modern urban tramway vehicles are low floor, they feature bidirectional movement, and their frame is constructed with rounded edges in order to protect pedestrians in the event of a collision.

To improve vehicle accessibility for people with disabilities, the rolling stock manufacturers and the network operators have adopted various measures such as

- Construction of high platforms
- General reduction of the height of the vehicle floor
- Reduction of the floor height only at the door ascending/descending spaces
- Addition of an extra car with lower floor height

The number of doors of a tram must necessarily be more than three (4–8), with a minimum width of 70 cm in the case of a single door and 1.20 m in the case of a double door. The height of the doors must necessarily be greater than 1.85 m. Hence, the passengers are better served at stops, and halt times are reduced (boarding-alighting time of about 20 sec).

The bogies of tramway vehicles must allow for the successfull inscription of vehicles in curved track segments that have very small radii (up to 20–25 m), while at the same time they must be able to top speeds of 80–90 km/h in straight track segments (of track of good ride quality). In this context, the bogies of modern tramway vehicles are different from the bogies used in other railway systems. More specifically, two alternative technologies of bogies are currently used, namely (Pyrgidis, 2004; Pyrgidis and Panagiotopoulos, 2012):

- Bogies with independently rotating wheels
- Bogies with mixed behaviour (they operate as conventional bogies at straight paths of the track, and as bogies with independently rotating wheels in curves)

Regardless of the bogie's technology, in order to optimise their characteristics so as to meet the track alignment geometric data, and to achieve the desired performance, manufacturers may adopt (see paragraph 3.2.4)

- A small wheel base ($2a = 1.70–2.00$ m instead of $2.50–3.00$ m which is applied in conventional railway vehicles)
- A small wheel diameter ($2r_o = 0.60–0.70$ m instead of $0.80–1.00$ m which is applied in conventional railway vehicles)

The equivalent conicity of the wheels and the stiffness of the primary suspension springs constitute critical parameters of the bogies' construction for their lateral behaviour (see paragraph 3.2.4).

The gradual braking (service braking) of tramway trains is usually ensured by an electric braking system (rheostat braking system equipped with energy regeneration system) which, in a second phase, is replaced by mechanical braking (brake discs, pads). The transmission is either pneumatic or electrical. The emergency braking is ensured by the additional effect of electromagnetic braking.

4.3.3 Tramway signalling system and traffic control

The basic principles of tram signalling systems are that (a) priority at traffic signal locations should be given to trams, and (b) at level crossings there should be collaboration between the different signalling systems intended for trams, road vehicles and pedestrians.

Regarding priority for trams at traffic lights, there are two strategies:

Passive Traffic Signal Priority: In these systems, traffic lights are set to turn green based on an average tram speed. In other words, the detection of a tram at crossings with traffic lights is not necessary. Priority is given by a standard procedure: favourable cycle time – favourable green time at each phase of the cycle time-coordination.

Active Traffic Signal Priority: In this strategy, the approaching tram sends a signal to the traffic signal controller which can change the signal, within predefined limits, in its favour. Active traffic signal priority is more effective than passive traffic signal priority, as it is based on a dynamic response to a transit request.

There are four types of active traffic signal priority systems for tramway systems:

- Dedicated priority by phasing changes
- Priority by extended green time
- Priority by phase and phase timing adjustment
- Implementation of Intelligent Transport Systems approaches

In order to locate the tram's position, two different approaches are applied: the use of Global Positioning Satellite system, or the use of special sensors which are placed on the pavement and can detect the tram as it passes over them.

The literature states that tram priority at intersections can increase the commercial speed of tram-trains by 35% (Pyrgidis and Chatziparaskeva, 2012; Foox et al., No date).

4.3.4 Transport capacity of the system

The majority of tramway systems transport 150–250 passengers per train (standing and seated) (Lesley, 2011). In any event, the capacity of the tram, for a given acceptable passenger density (e.g., 6 passengers/m^2, 4 passengers/m^2), may be calculated by taking into consideration the inner length and width of the vehicle and the number of passenger seats.

Figure 4.15 (Bieber, 1986) presents average transport system capacity values in relation to the trains' headway for four types of trams and two types of urban buses (considering average train/vehicle capacity values). The diagram is restricted to service frequencies between 6 and 60 trains/h. Frequencies greater than 60 trains/h cause problems to other traffic and reduce the level of service.

According to the literature, the headway of two successive trains ranges from 1.5 min up to 30 min, depending on the volume of passenger traffic, the day, and the hour of the day. The typical values range from 5 min (peak hours) to 15 min (off-peak hours). The lowest values of train headways are recorded for the Hong Kong tram system (1.5 min during peak hours).

Regarding the passenger traffic volume, Table 4.2 presents the annual passenger traffic per network km for tramway systems of all European countries (ERRAC-UITP, 2009). Table 4.3 lists the results of statistical analysis on the annual ridership per network km in relation to a city's population.

4.3.5 Travel time and commercial speeds

The travel time is one of the fundamentals that determines the quality of service provided by a tramway network to its users. Low values of travel time render the tram a more attractive transportation mode and bring about a significant increase to its potential transportation volume.

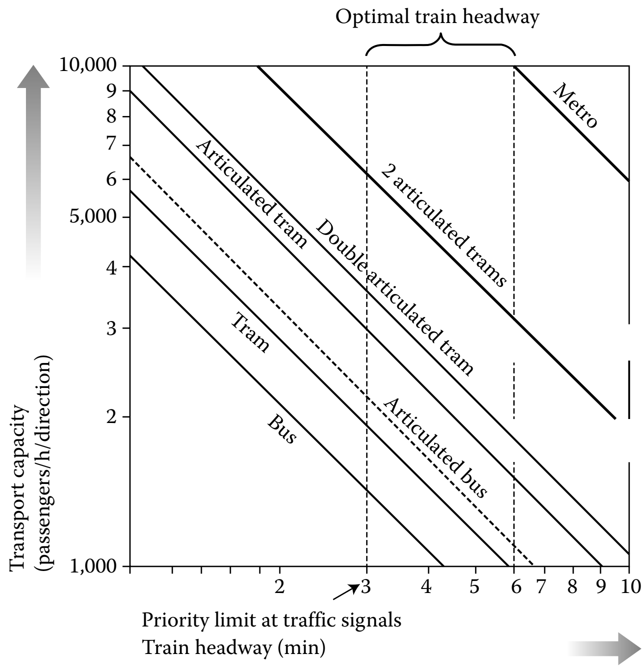

Figure 4.15 Train headway – transport capacity chart for urban public transport modes. (Adapted from Bieber, C. A. 1986, *Les choix techniques pour les transports collectifs*, Lecture Notes, Ecole Nationale des Ponts et Chaussées, Paris.)

Table 4.2 Annual ridership per network km for tramway systems in European countries

Country	*Million passengers/network km/year (average values)*
EU-15	1.00
Recently acceded countries	1.97
Acceding countries	1.36

Source: Adapted from ERRAC-UITP 2009, Metro, light rail and tram systems in Europe.

Table 4.3 Annual ridership per network km in relation to a city's population

City's population (passengers) (ppl)	*Million passengers/network km/year (average values)*
ppl ≤ 300,000	1
300,000 < ppl ≤ 900,000	1.3
900,000 < ppl ≤ 1,5000,00	1.6
1,500,000 < ppl ≤ 1,800,000	1.8
>5,000,000 ppl	3.8 (Hong Kong, 6 million passengers)

Table 4.4 Percentage of protected tramway corridors in Europe

Country	*Percentage of the length of the tram corridor that is protected (classes A, B, D) (%)*
EU-15	74
Recently acceded countries	63
Acceding countries	87

Source: Adapted from ERRAC-UITP 2009, Metro, light rail and tram systems in Europe.

The value of travel time and, consequently, the average operating speed (or otherwise commercial speed) of a tram line, depends upon various parameters, such as (Pyrgidis et al., 2013):

- The category of tramway corridors encountered along the route and the length per category
- The signalling system applied at level crossings with where the tram intersects with other transportation modes
- The number of intermediate stops and the distance between them
- The halt time
- The citizens' awareness of the presence of trams and their attitude
- The tram drivers' behaviour
- The track alignment
- The performance of the rolling stock
- The number of level crossings
- The time of the day

The most important parameters are the category of the tramway corridor and the signalling system used.

The proportion of the length of the tramway corridor that is protected (classes A, B, D) to the total length of the tram route is usually high and ranges between 60% (e.g., in Zurich) and 90%–100% (e.g., in Cologne, Karlsruhe). Table 4.4 lists the average protection percentages for all tramway networks in Europe (ERRAC-UITP, 2009).

With regard to commercial speeds, values between $V_c = 13.5$–35 km/h have been recorded in practice at various low-floor tram networks, while the most usual values are between $V_c = 15$–25 km/h.

Table 4.5 lists the commercial speeds that are developed by average at all tramway networks in Europe (ERRAC-UITP, 2009).

4.3.6 Cost of implementing a tramway

For the assessment of the construction cost of tramway lines, the most recent urban low-floor systems were selected (2012–2014 data). The power supply system, labour costs and the extent of any underground or extensive surface civil engineering works/rehabilitation are key parameters that differentiate the total implementation cost.

The urban low-floor tramway systems cost between €14 M (Palermo) up to over €60 M per track-km.

The average total cost of construction per km of an urban tramway with overhead power supply system is in the range of €20–23.5 M (€22.5 M for Europe, €20 M for Africa and

Table 4.5 Average commercial speed of all tramway systems of European countries

Country	*Commercial speed (average values) km/h*
EU-15	22.6
Recently acceded countries	15.71
Acceding countries	21.10

Source: Adapted from ERRAC-UITP 2009, Metro, light rail and tram systems in Europe.

€23.5 M for North America). Any special civil engineering works may significantly increase these costs (e.g., €62.8 M for the Jerusalem tram, €84.5 M for the Seattle tram).

Table 4.6 gives the total implementation cost per track-km for the five of the six in total in service tramway systems using mixed power supply technology (overhead catenary system + ground power supply system (APS)).

In Dubai, the APS system was applied to the total length of the network (10 km). The overall construction cost of the system reached €66.8 M per km (2014 data) due to the exclusive use of the APS, but mainly due to the high manufacturing cost of stations (indoor areas and air-conditioning).

The average cost per vehicle for an urban tram with overhead catenary system amounts in the range of €2.5 M (width 2.40 m length 32 m). This cost can reach up to €3.5 million or even more depending on the available equipment and the capabilities of the vehicle.

The average cost for an urban tram vehicle with a prower supply system at ground level is approximately 15% bigger.

For tram–trains, the cost of a vehicle is significantly higher, and it approximates €4 M–€4.5 M.

4.4 INTEGRATION OF TRAMWAY CORRIDORS ACROSS THE ROAD ARTERIES

4.4.1 Types of integration of tramway corridors

The integration of at-grade tramway lines within the right of way can take place in different ways depending on the geometric and traffic characteristics of the road and the nature of the roadside land uses. More in particular

Table 4.6 Total implementation cost of tramway systems using mixed power supply technology (overhead catenary system + ground power supply system (APS))

City/France	*Line length (km)*	*Percentage of line length with APS (%)*	*Cost per km (million €)*
Tours	14.8	13.5	27.8
Orleans	11.8	21.2	30.8
Angers	12	12.5	32.9
Reims	11	18.2	34.1
Bordeaux	44	27.3	35.7

Figure 4.16 Placement of a single tramway track at the two opposite sides of the roadway. (From Thessaloniki Public Transport Authority, 2015.)

4.4.1.1 A single track per direction at two opposite sides of the road

This option is more appropriate for roads that operate as one-way roads for all other traffic.

Its main advantages are as follows (Figure 4.16):

- In the case of an overhead power supply system, masts can be installed on the side footpaths. As a result a smaller right-of-way is required.
- At the stop areas, the existing sidewalks may well serve as part of the platforms.

Its main disadvantages are as follows:

- A noticeable difficulty in feeding the adjacent land uses, which necessarily takes place during specific hours of the day, especially during hours when the tram stops operating or operates with very low frequencies.
- In the case of small building blocks, the increased number of intersections can reduce the travel time savings, which come as a result of the segregation of the tram from all other traffic.

4.4.1.2 Double track on one side of the road

In this case, in the vicinity of the tramway stops, it is required to build an islet in order to create a platform for the second vehicle. As a result, the road width is reduced. This problem can be solved by creating a recess, which, however, reduces the width of the sidewalk (Figure 4.17).

This is the simplest and least space-consuming integration, however, it has some negative impact on the local residents and their activities. In the case of small building blocks, the successive intersections with right-turning movements can reduce the travel time savings, which come as a result from the segregation of the tram from all other traffic.

4.4.1.3 Central alignment

The tramway system is located in the centre of the right-of-way, usually in double track superstructure (Figure 4.18). With this integration, there is no problem with turning road vehicles movements. In the case of overhead power supply, the integration of the tram in the centre of the road artery may be implemented in two ways:

Figure 4.17 Placement of a double tramway track on one side of the roadway. (From Thessaloniki Public Transport Authority, 2015.)

- *Without electrictification masts between the two tracks:* In this case, the power supply of tramway vehicles is achieved through wires, which are suspended on laterally-opposed masts or connected to building facades. This integration offers a smaller right-of-way.
- *With electrification masts placed centrally between the two tracks (Figure 4.18):* This solution is preferable in terms of aesthetics, but it is more expensive and least favourable for the road traffic as it requires greater corridor width.

The main advantage of placing the tramway system in the centre of the road is the ease of access and feeding of the adjacent land uses, especially in the case of two-way traffic roads where positioning the tramway system at the two opposite sides of the road would significantly impede their operations.

The main disadvantage of placing the tramway system in the centre of the road is the risk regarding the crossing of the rest of the road by pedestrians. In order to alleviate this risk, the construction of an islet with a width of 2.0 m, is required at stops to ensure the comfortable and safe boarding and alighting of passengers. This, however, has an obvious negative effect, namely the reduction of the road width at these locations.

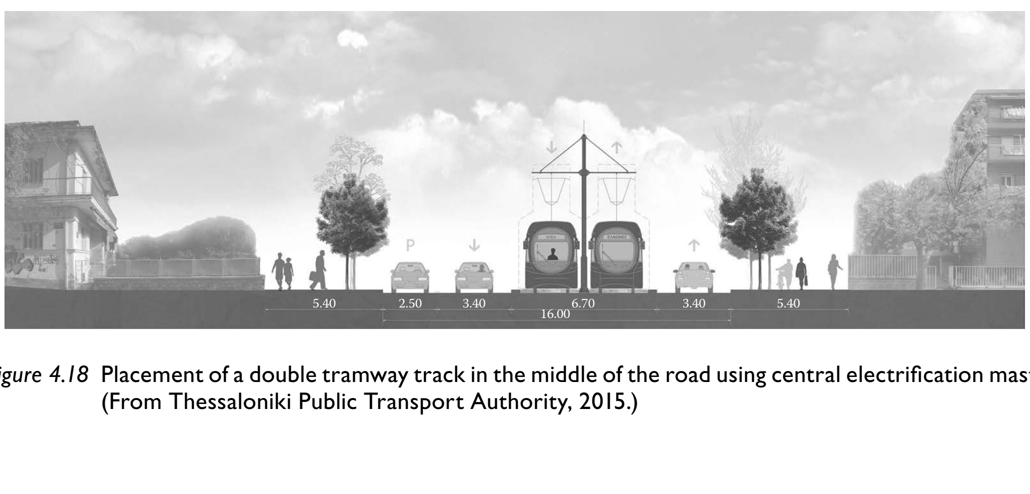

Figure 4.18 Placement of a double tramway track in the middle of the road using central electrification mast. (From Thessaloniki Public Transport Authority, 2015.)

4.4.2 Geometric features of the integration of tramway corridors

In order to decide on the integration type of the selected categories of tramway corridors (classes A, B, C, D, E) it is essential to investigate the adequacy of the geometric features of the road in relation to the geometric features which are required for each case for the operation of the tramway system.

4.4.2.1 Technical and total tramway infrastructure right-of-way

The term *technical right-of-way* describes the minimum width required for the safe operation of the tramway system. For line segments between stops (the 'plain line') technical right-of-way is defined by the number of tracks, their gauge, the width of the dynamic gauge of tramway vehicles and the civil engineering structure gauge.

Depending on how the tram is integrated across the road artery, the technical right-of-way of the tramway system should be increased either on both sides or on one side only, by such a distance that allows for the installation of seperators between the tram and the rest of the traffic.

The final resulting width is called Total Tramway infrastructure technical Right-Of-Way (TTROW). Concrete bollards with a height of 15–20 cm, and a width of 40 cm or greater (minimum width 30 cm) where plants are dibbled may be used as means of segregation. These bollards may also be used for posting signs or signals, which will serve the signalling not only of the tramway system but also of other traffic. If the width of these bollards is greater than 1.20 m they can be used as intermediate stops for pedestrians during their movement on the level crossing. Apart from these segregation means, hatched lane (with a width of 40 cm), railings, wall separators, trees and so on, can also be used, depending on the type of integration of the tramway system in the road.

4.4.2.1.1 Total Tramway infrastructure technical Right-Of-Way at Straight segments of the alignment (TTROWS)

In the case of double track, at straight paths, the structure gauge is equal to the dynamic gauge of vehicles increased by 100 mm at either side of the double track, and by 200 mm between opposite moving tram vehicles.

The mathematical relationships that can be used in order to calculate the TTROWS for different ways of integrating tramway corridors across the road arteries, and depending on the category of tramway corridor, are shown hereunder.

1. Placement of a double tramway track at the centre of the roadway (central alignment)

$$\mathrm{TTROWS} = 2 \times (b_{sw} + 0.1 + g_{dv}) + b_{em} + 0.2 \quad \text{(classes A, B, C, D)} \tag{4.1}$$

$$\mathrm{TTROWS} = 2 \times g_{dv} + b_{em} + 0.4 \quad \text{(class E)} \tag{4.2}$$

whereas

b_{sw}: Width of separator
g_{dv}: Dynamic gauge width of tram vehicle
b_{em}: Width needed for the installation of electrification masts

In case no electrictification mast is foreseen, $b_{em} = 0$

2. Placement of a double tramway track on one side of the roadway

$$\text{TTROWS} = 2 \times (0.1 + g_{dv}) + b_{em} + b_{sw} + 0.2 \quad \text{(classes A, B, C, D)} \tag{4.3}$$

$$\text{TTROWS} = 2 \times g_{dv} + b_{em} + 0.4 \quad \text{(class E)} \tag{4.4}$$

In case no electrictification mast is foreseen, $b_{em} = 0$

3. Placement of a single tramway track at the two opposite sides of the roadway

$$\text{TTROWS} = 2 \times (b_{sw} + 0.1 + 0.1 + g_{dv}) \quad \text{(classes A, B, C, D)} \tag{4.5}$$

$$\text{TTROWS} = 2 \times (0.2 + g_{dv}) \quad \text{(class E)} \tag{4.6}$$

Table 4.7 provides the minimum values of the total tramway infrastructure right-of-way which are required in order to integrate a tramway system across a road artery for a vehicle width of 2.30 m (dynamic gauge width $g_{dv} = 2.60$ m), for a separator with a width of $b_{sw} = 0.40$ m, for an electrictrification mast installation width of $b_{em} = 0.30$ m, and for corridor categories A, B, C, D.

4.4.2.1.2 Total Tramway Infrastructure Right-Of-Way in Curves (TTROWC)

A larger right-of-way is required in curves, and this is due to the following reasons:

- Primarily, it is due to the extra space required, because of the geometry, for the integration of the tracks across the road.

  This extra space depends on the angle between the two intersecting roads and on the type of integration of the tramway line across the road before and after the intersection. In this context the required right-of-way of the tramway lines can be related and be expressed via the minimum required width of intersecting roads (Chatziparaskeva and Pyrgidis, 2015).
- Secondly, it is due to the effects of vehicle end throw on the outside of a curve and centre throw on the inside of a curve (Figures 4.19 and 4.20) (Mundrey, 2000). These

Table 4.7 Total infrastructure right-of-way values for a tramway system straight path

Types of integration of tramway corridors across the road	*Straight path (TTROWS) (m)*
Placement at one side of the roadway–single track	3.20
Placement at one side of the roadway–double track without electrification mast	6.00
Placement at one side of the roadway–double track with a centrally placed electrictrification mast	6.30
Placement at the centre of the roadway–double track without a centrally placed electritrification mast	6.40
Placement at the centre of the roadway–double track with a centrally placed electrictrification mast	6.70
Placement at the two opposite sides of the roadway–single track	6.40

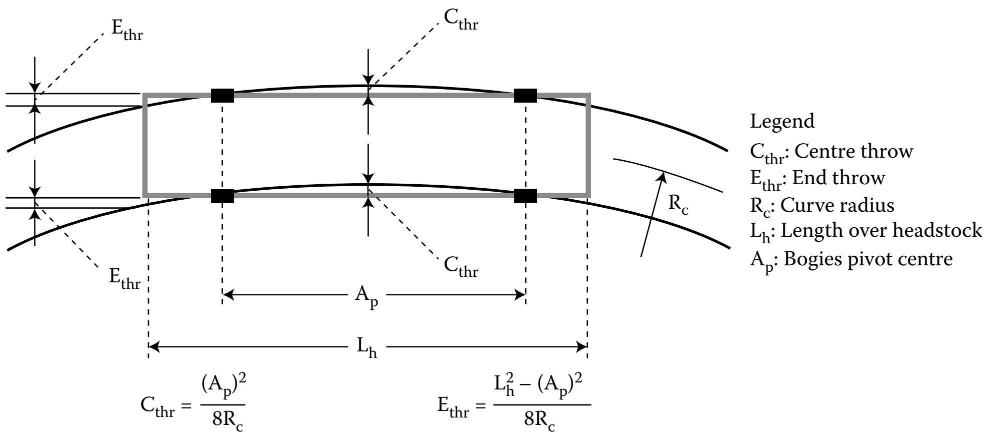

Figure 4.19 Inscription of a railway vehicle in curves. Definition of end/centre throw. (Adapted from CRN CS 215 2013, Engineering Standard Track, version 1.1, July 2013.)

reach their greatest values in the middle of the circular arc of the curved segment. They directly depend on the length of the cars of the articulated tramway vehicle (they increase as the length increases).

Engineers can estimate the right-of-way at turns through appropriate design simulation.

4.4.2.2 Geometric integration of tramway corridors at curved sections of roads in the horizontal alignment

The geometric integration of a tramway corridor in curved sections of the right-of-way requires the horizontal alignment of the tram, including considerations for end and centre throw, to be able to match the horizontal alignment of the road artery and space

Figure 4.20 Inscription of a double articulated tramway vehicle in curves. Effects of vehicle end and centre throw.

constraints of the right of way in general; a minimum radius of horizontal alignment equal to R_{cmin} = 25 m is the usual minimum value considered.

A design simulation for all possible combinations regarding the integration type of the tramway track before and after the intersection, for intersection angles that vary between $\varphi_o = 90°$ and $\varphi_o = 170°$ has been developed in the literature (Chatziparaskeva and Pyrgidis, 2015). For this simulation, the following were considered:

- Static vehicle width equal to 2.30 m and dynamic vehicle width equal to 2.60 m
- Curve radius in the horizontal alignment R_c = 25 m (minimum permitted)
- Integration without central electrictrification mast
- Right turn and left turn
- At the area of the turn, the tramway corridor is common (class E)

The design simulation provided the minimum required road widths b1 and b2 of the road arteries. Table 4.8 presents the indicative results for all possible combinations of integration of the tramway track, for intersection angle $\varphi_o = 120°$.

Regarding the symbols used for the integration type (column 1 of Table 4.8):

- The first letter indicates whether the corridor is exclusive for trams or whether the use by other road vehicles is also permitted (classes B, D). More specifically, the following symbols are adopted:
  C: Corridor class C (exclusive tramway corridor without use by other road vehicles)
  F: Corridor class B and D (protected or separated tramway corridors)
- The second letter indicates the integration type of the double tramway track. More specifically the following symbols are adopted:
  A: Integration of the track at the left side of the road
  Δ: Integration of the track at the right side of the road
  K: Integration of the track in the middle of the road
- The number 1 refers to single track, whereas the number 2 refers to double track
- The letter α indicates left turn movement, whereas the letter δ indicates right turn movement
- Finally, the symbol X indicates that integration is not possible

As an example, the symbols F2Δ-F2K (δ) indicate transition by a right turn, from a tramway corridor category B or D, with a double track placed at the right side of the roadway to a tramway corridor category B or D, with a double track placed at the centre of the roadway (Figure 4.21).

In order to enable the geometric integration of the tramway tracks at turns, even with the smallest allowable radius of 25 m, the two intersecting roads must have the available width that is calculated by following the procedure described above.

4.5 INTEGRATION OF STOPS

4.5.1 Types of stops integration

Terminals and stops constitute an important component of the tramway system. They are considered as structural elements of the tramway infrastructure and they fall under its operational facilities. Their presence in the system is necessary because they allow for the boarding and alighting of passengers to/from the trains.

Table 4.8 Integration of a double tramway track in curved sections of the roads in the horizontal alignment – Required roadway width for road intersection angle $\varphi_o = 120°$

Road width/ Integration type	*b1 = 7 m*	*b1 = 8 m*	*b1 = 9 m*	*b1 = 10 m*	*b1 = 11 m*	*b1 = 12 m*	*b1 = 13 m*	*b1 = 14 m*	*b1 = 15 m*	*b1 = 16 m*	*b1 = 17 m*	*b1 = 18 m*	*b1 = 19 m*	*b1 = 20 m*
CA-CA (δ) CΔ-CΔ (α)	X	b1 = 8 b2 = 13.02	b1 = 9 b2 = 11.26	b1 = 10 b2 = 10.09	b1 = 11 b2 = 9.2	b1 = 12 b2 = 8.52	b1 = 13 b2 = 8.01	b1 = 14 b2 = 7.61	b1 = 15 b2 = 7.32	X	X	X	X	X
CA-CK (δ) CΔ-CK (α)	X	X	X	b1 = 10 b2 = 13.78	b1 = 11 b2 = 12	b1 = 12 b2 = 10.64	b1 = 13 b2 = 9.62	b1 = 14 b2 = 8.82	b1 = 15 b2 = 8.24	X	X	X	X	X
CA-CΔ (δ) CΔ-CA (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
CK-CA (δ) CK-CΔ (α)	X	b1 = 8 b2 = 15.52	b1 = 9 b2 = 13.75	b1 = 10 b2 = 12.6	b1 = 11 b2 = 11.71	b1 = 12 b2 = 10.99	b1 = 13 b2 = 10.39	b1 = 14 b2 = 9.88	b1 = 15 b2 = 9.43	X	X	X	X	X
CK-CK (δ) CK-CK (α)	X	X	X	X	X	b1 = 12 b2 = 15.58	b1 = 13 b2 = 14.38	b1 = 14 b2 = 13.36	b1 = 15 b2 = 12.46	X	X	X	X	X
CK-CΔ (δ) CK-CA (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
CΔ-CA (δ) CA-CΔ (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
CΔ-CK (δ) CA-CK (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
CΔ-CΔ (δ) CA-CA (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
CA-FA (δ) CΔ-FΔ (α)	b1 = 7 b2 = 16.69	b1 = 8 b2 = 12.62	b1 = 9 b2 = 10.86	b1 = 10 b2 = 9.69	b1 = 11 b2 = 9	b1 = 12 b2 = 9	b1 = 13 b2 = 9	b1 = 14 b2 = 9	b1 = 15 b2 = 9	X	X	X	X	X
CA-FK (δ) CΔ-FK (α)	b1 = 7 b2 = 27.78	b1 = 8 b2 = 19.64	b1 = 9 b2 = 16.12	b1 = 10 b2 = 13.78	b1 = 11 b2 = 12.4	b1 = 12 b2 = 12.4	b1 = 13 b2 = 12.4	b1 = 14 b2 = 12.4	b1 = 15 b2 = 12.4	X	X	X	X	X
CA-FΔ (δ) CΔ-FA (α) CA-F1F2 (δ) CΔ-F1F2 (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
CK-FA (δ) CK-FΔ (α)	b1 = 7 b2 = 18.21	b1 = 8 b2 = 15.12	b1 = 9 b2 = 13.35	b1 = 10 b2 = 12.4	b1 = 11 b2 = 11.31	b1 = 12 b2 = 10.59	b1 = 13 b2 = 9.99	b1 = 14 b2 = 9.48	b1 = 15 b2 = 9.03	X	X	X	X	X
CK-FK (δ) CK-FK (α)	b1 = 7 b2 = 30.82	b1 = 8 b2 = 24.64	b1 = 9 b2 = 21.1	b1 = 10 b2 = 19.2	b1 = 11 b2 = 17.02	b1 = 12 b2 = 15.58	b1 = 13 b2 = 14.38	b1 = 14 b2 = 13.36	b1 = 15 b2 = 12.46	X	X	X	X	X

(*Continued*)

Table 4.8 (Continued) Integration of a double tramway track in curved sections of the roads in the horizontal alignment – Required roadway width for road intersection angle $\varphi_o = 120°$

Road width/ Integration type	*b1 = 7 m*	*b1 = 8 m*	*b1 = 9 m*	*b1 = 10 m*	*b1 = 11 m*	*b1 = 12 m*	*b1 = 13 m*	*b1 = 14 m*	*b1 = 15 m*	*b1 = 16 m*	*b1 = 17 m*	*b1 = 18 m*	*b1 = 19 m*	*b1 = 20 m*
CK-FΔ (δ) CK-FA (α) CK-F1F2 (δ · α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
CΔ-FA (δ) CA-FΔ (α)	b1 = 7 b2 = 18.11	b1 = 8 b2 = 18.11	b1 = 9 b2 = 18.11	b1 = 10 b2 = 18.11	b1 = 11 b2 = 18.11	b1 = 12 b2 = 18.11	b1 = 13 b2 = 18.11	b1 = 14 b2 = 18.11	b1 = 15 b2 = 18.11	X	X	X	X	X
CΔ-FK (δ) CA-FK (α)	b1 = 7 b2 = 30.82	b1 = 8 b2 = 30.82	b1 = 9 b2 = 30.82	b1 = 10 b2 = 30.82	b1 = 11 b2 = 30.82	b1 = 12 b2 = 30.82	b1 = 13 b2 = 30.82	b1 = 14 b2 = 30.82	b1 = 15 b2 = 30.82	X	X	X	X	X
CΔ-FΔ (δ) CA-FA (α) CΔ-F1F2 (δ) CA-F1F2 (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
FA-CA (δ) FΔ-CΔ (α)	X	X	b1 = 9 b2 = 10.74	b1 = 10 b2 = 9.69	b1 = 11 b2 = 8.91	b1 = 12 b2 = 8.3	b1 = 13 b2 = 7.84	b1 = 14 b2 = 7.48	b1 = 15 b2 = 7.23	b1 = 16 b2 = 7.05	b1 = 17 b2 = 6.94	b1 = 18 b2 = 6.9	b1 = 19 b2 = 6.4	b1 = 20 b2 = 6.4
FA-CK (δ) FΔ-CK (α)	X	X	b1 = 9 b2 = 15.08	b1 = 10 b2 = 12.98	b1 = 11 b2 = 11.42	b1 = 12 b2 = 10.2	b1 = 13 b2 = 9.28	b1 = 14 b2 = 8.56	b1 = 15 b2 = 8.06	b1 = 16 b2 = 7.7	b1 = 17 b2 = 7.48	b1 = 18 b2 = 7.4	b1 = 19 b2 = 6.4	b1 = 20 b2 = 6.4
FA-CΔ (δ) FΔ-CA (α)	X	X	X	X	X	X	X	X	X	X	X	X	b1 = 19 b2 = 6.4	b1 = 20 b2 = 6.4
FK-CA (δ) FK-CΔ (α)	X	X	X	X	X	X	b1 = 13 b2 = 10.39	b1 = 14 b2 = 9.88	b1 = 15 b2 = 9.43	b1 = 16 b2 = 9.05	b1 = 17 b2 = 8.71	b1 = 18 b2 = 8.41	b1 = 19 b2 = 8.15	b1 = 20 b2 = 7.92
FK-CK (δ) FK-CK (α)	X	X	X	X	X	X	b1 = 13 b2 = 14.38	b1 = 14 b2 = 13.36	b1 = 15 b2 = 12.46	b1 = 16 b2 = 11.7	b1 = 17 b2 = 11.02	b1 = 18 b2 = 10.42	b1 = 19 b2 = 9.9	b1 = 20 b2 = 9.44
FK-CΔ (δ) FK-CA (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
FΔ-CA (δ) FA-CΔ (α) F1F2-CA (δ) F1F2-CΔ (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
FΔ-CK (δ) FA-CK (α) F1F2-CK(δ · α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X

(Continued)

Table 4.8 (Continued) Integration of a double tramway track in curved sections of the roads in the horizontal alignment – Required roadway width for road intersection angle $\varphi_o = 120°$

Road width/ Integration type	*b1 = 7 m*	*b1 = 8 m*	*b1 = 9 m*	*b1 = 10 m*	*b1 = 11 m*	*b1 = 12 m*	*b1 = 13 m*	*b1 = 14 m*	*b1 = 15 m*	*b1 = 16 m*	*b1 = 17 m*	*b1 = 18 m*	*b1 = 19 m*	*b1 = 20 m*
FΔ-CΔ (δ) FA-CA (α) F1F2-CΔ (δ) F1F2-CA (α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
FA-FA (δ) FΔ-FΔ (α)	X	X	b1 = 9 b2 = 10.34	b1 = 10 b2 = 9.29	b1 = 11 b2 = 9	b1 = 12 b2 = 9	b1 = 13 b2 = 9	b1 = 14 b2 = 9	b1 = 15 b2 = 9	b1 = 16 b2 = 9	b1 = 17 b2 = 9	b1 = 18 b2 = 9	b1 = 19 b2 = 9	b1 = 20 b2 = 9.0
FA-FK (δ) FΔ-FK (α)	X	X	b1 = 9 b2 = 15.08	b1 = 10 b2 = 12.98	b1 = 11 b2 = 12.4	b1 = 12 b2 = 12.4	b1 = 13 b2 = 12.46	b1 = 14 b2 = 12.4	b1 = 15 b2 = 12.4	b1 = 16 b2 = 12.4	b1 = 17 b2 = 12.4	b1 = 18 b2 = 12.4	b1 = 19 b2 = 12.4	b1 = 20 b2 = 12.4
FA-FΔ (δ) FΔ-FA (α) FA-F1F2 (δ) FΔ-F1F2 (α)	X	X	X	X	X	X	X	X	X	X	X	X	b1 = 19 b2 = 9 b2 = 9.4 (F1F2)	b1 = 20 b2 = 9.0 b2 = 9.4 (F1F2)
FK-FA (δ) FK-FΔ (α)	X	X	X	X	X	X	b1 = 13 b2 = 9.99	b1 = 14 b2 = 9.48	b1 = 15 b2 = 9.03	b1 = 16 b2 = 9	b1 = 17 b2 = 9	b1 = 18 b2 = 9	b1 = 19 b2 = 9	b1 = 20 b2 = 9
FK-FK (δ) FK-FK (α)	X	X	X	X	X	X	b1 = 13 b2 = 14.38	b1 = 14 b2 = 13.36	b1 = 15 b2 = 12.46	b1 = 16 b2 = 12.4	b1 = 17 b2 = 12.4	b1 = 18 b2 = 12.4	b1 = 19 b2 = 12.4	b1 = 20 b2 = 12.4
FK-FΔ (δ) FK-FA (α) FK-F1F2 (δ · α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X
FΔ-FA (δ) FA-FΔ (α) F1F2-FA (δ) F1F2-FΔ (α)	X	X	b1 = 9 b2 = 18.11 X (F1F2)	b1 = 10 b2 = 18.11	b1 = 11 b2 = 18.11	b1 = 12 b2 = 18.11	b1 = 13 b2 = 18.11	b1 = 14 b2 = 18.11	b1 = 15 b2 = 18.11	b1 = 16 b2 = 18.11	b1 = 17 b2 = 18.11	b1 = 18 b2 = 18.11	b1 = 19 b2 = 18.11	b1 = 20 b2 = 18.11
FΔ-FK (δ) FA-FK (α) F1F2-FK (δ · α)	X	X	b1 = 9 b2 = 30.82 X (F1F2)	b1 = 10 b2 = 30.82	b1 = 11 b2 = 30.82	b1 = 12 b2 = 30.82	b1 = 13 b2 = 30.82	b1 = 14 b2 = 30.82	b1 = 15 b2 = 30.82	b1 = 16 b2 = 30.82	b1 = 17 b2 = 30.82	b1 = 18 b2 = 30.82	b1 = 19 b2 = 30.82	b1 = 20 b2 = 30.82
FΔ-FΔ (δ) FA-FA (α) F1F2-F1F2 (δ · α)	X	X	X	X	X	X	X	X	X	X	X	X	X	X

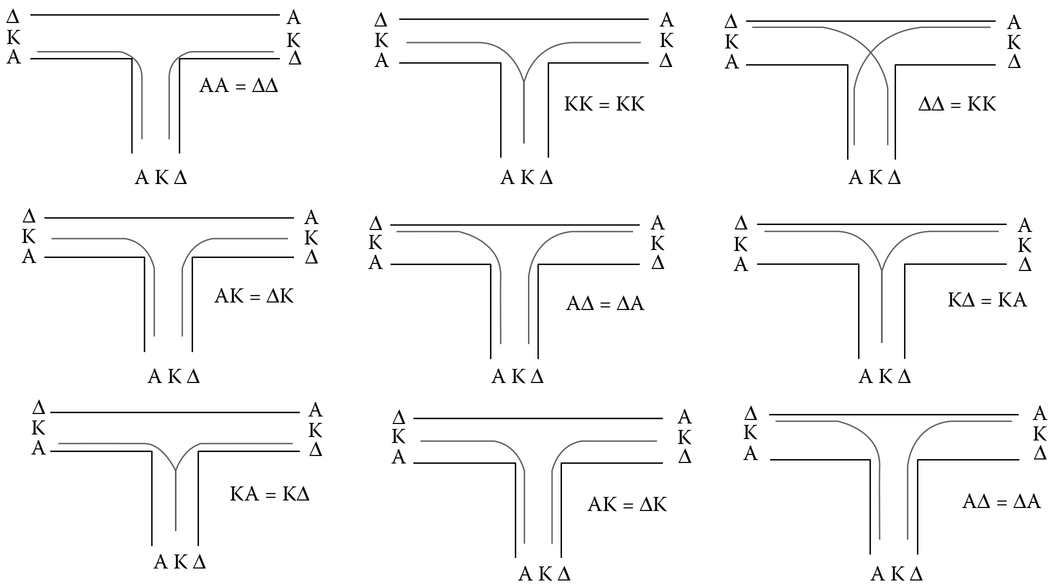

Figure 4.21 Types of integration of double track tramway corridors across the road before and after the turn.

Three categories of tramway stops may be considered (RATP.D.D.E, 1994):

- Stop with centre (island) platform (Figure 4.22)
- Stop with laterally staggered platforms (Figure 4.23)
- Stop with laterally opposed platforms (Figure 4.24)

The level of service provided to the users of a tramway system at stops is determined by the degree to which certain parameters are satisfied. The key parameters (quality parameters) which reflect the user needs are

- The acceptable distance between successive stops
- The location of stops at areas where land uses constitute attractors of a large number of trips

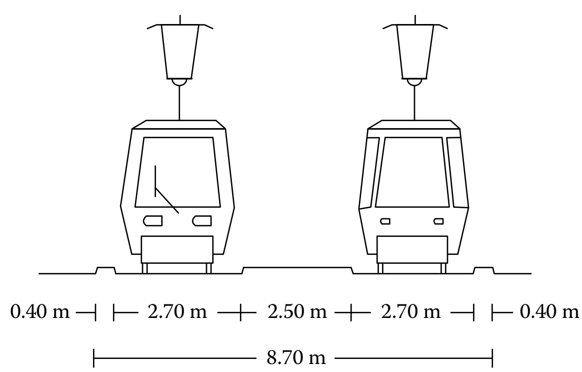

Figure 4.22 Stop with centre (island) platform. (Adapted from RATP.D.D.E 1994, Projet de rocade tramway en site propre entre Saint Denis et Bobigny: Schéma de principe, 1993, Paris, Février.)

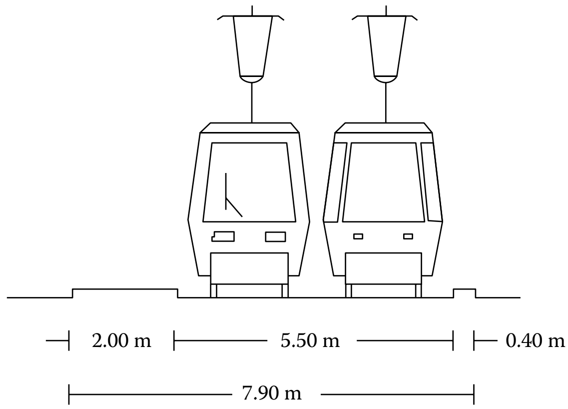

Figure 4.23 Stop with laterally staggered platforms. (Adapted from RATP.D.D.E 1994, Projet de rocade tramway en site propre entre Saint Denis et Bobigny: Schéma de principe, 1993, Paris, Février.)

- The required halt time
- The accessibility of the stop
- The ability for quick transfer to other modes of public transport
- The service of people with reduced mobility
- The safety and comfort of passengers while waiting at the stops (seats)
- The information regarding the route and the next train arrival available for passengers
- The easy supply of tickets
- The interfaces between staff and users
- The easy identification of the stop from afar
- The protection of users against bad weather conditions (shelter)
- The aesthetics of the stop
- The attractiveness of the stop (surface integration, location at areas with recreational activities, retail or medical facilities)

A questionnaire survey among tram users in Athens revealed that the most important parameter for a tram stop is the available services in terms of land uses (preference 43%),

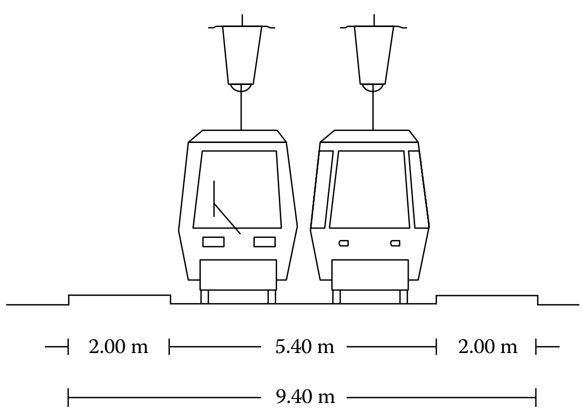

Figure 4.24 Stop with laterally opposed platforms. (Adapted from RATP.D.D.E 1994, Projet de rocade tramway en site propre entre Saint Denis et Bobigny: Schéma de principe, 1993, Paris, Février.)

Figure 4.25 Tramway stop at Athens, Greece. (Photo: A. Klonos.)

followed by the ease of access to the stop (28%), and the adequacy of information and safety while waiting at the stop with percentages of preference equal to 12% and 11%, respectively (University of Pireaus, 2007).

Figures 4.25 and 4.26 illustrate a tram stop in the city of Athens, Greece, and Grenoble, France, respectively. This stop features a shelter, seats for the users, lighting, automatic ticketing machine, information display for the lines that serve the stop and waiting times in real time, route map, map of the area around the stop, closed circuit television(CCTV), separator between the stop and the other road traffic, accessibility ramp for the disabled and trash bin.

4.5.2 Geometric and operational features of tramway stop integration

The location of stops and the choice of the type of platforms for a tramway network is based on the following geometric and operational criteria.

4.5.2.1 Geometric criteria

In the area of Stops, the required Total Technical Right-Of-Way (TTROWST) is larger than the respective technical right-of-way for a 'plain' line, due to the presence of platforms.

The minimum allowed width for a central platform is 2.50 m, while the minimum allowed width for a side platform is 2.00 m (RATP.D.D.E, 1994).

The mathematical equations that can be used for the calculation of TTROWST for different types of integration of tram stops, depending on the category of tramway corridor are shown below.

1. Installation of a tramway stop of a double track with centre (island) platform

$$\text{TTROWST} = 2 \times (b_{sw} + 0.1 + g_{dv}) + b_{cp} \quad \text{(classes A, B, C, D)} \tag{4.7}$$

$$\text{TTROWST} = 2 \times (0.1 + g_{dv}) + b_{cp} \quad \text{(class E)} \tag{4.8}$$

where b_{cp}: width of the centre (island) platform opposed platforms

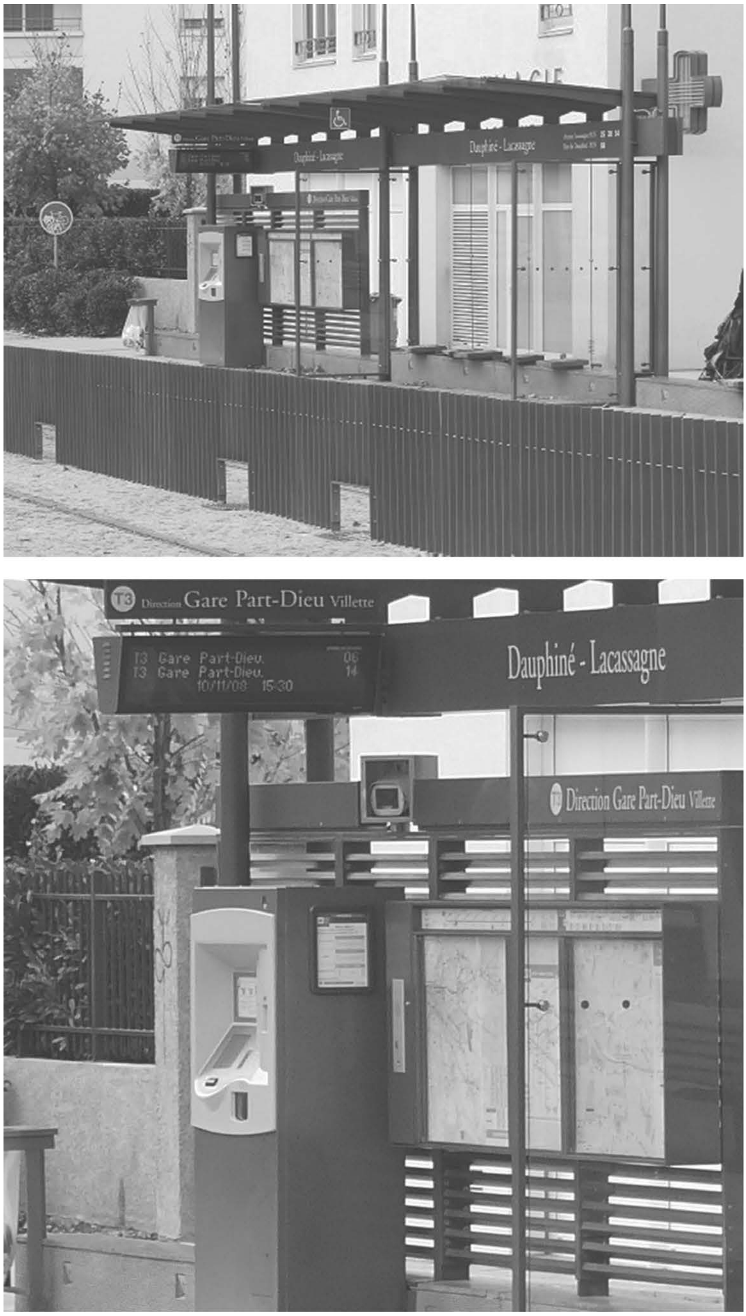

Figure 4.26 Tramway stop facilities, Lyon, France. (Adapted from Villetaneuse, C. 2008, available online at: https://fr.wikipedia.org/wiki/Ligne_3_du_tramway_de_Lyon (accessed 8 August 2015).)

2. Installation of a tramway stop of a double track with laterally opposed platforms

$$TTROWST = 2 \times (b_{lp} + g_{dv}) + 0.2 \quad \text{(classes A, B, C, D, E)} \tag{4.9}$$

where b_{lp}: width of the side platform

3. Installation of a tramway stop of a double track with laterally staggered platforms

$$TTROWST = b_{lp} + 2 \times g_{dv} + b_{sw} + 0.3 \quad \text{(classes A, B, C, D)} \tag{4.10}$$

$$TTROWST = b_{lp} + 2 \times g_{dv} + 0.3 \quad \text{(class E)} \tag{4.11}$$

The length of the platforms must allow for the stopping of the tram of the greatest length. Normally, and as long as is allowed by the length of the building blocks, the length of the platform should be at least double in order to allow for the stopping of two coupled trams during the peak hour.

4.5.2.2 Operational criteria

At intersections where full priority is granted to trams, it is preferable to install the stop after the intersection, so that the arrival time of the tram can be accurately estimated, as the stopping time largely depends on the time required for the boarding and alighting of passengers. This allows for the delay of the intersecting traffic to be minimised.

In case of an intersection where it is not desirable to give priority to trams, the most appropriate location of the stop is before the intersection, as the maximum delay time that can occur for the tram is equal to the length of the red phase of the light signal. Moreover, if the stopping time coincides with the red phase, there will be no delay.

For large uphill slopes it is preferable to place the tram stop at the end of the slope.

In case of curves with small radii, stops should preferably be located after the curve.

The distance between two successive stops should generally be greater than 400–450 m and less than 750–800 m.

4.6 TRAMWAY DEPOT FACILITIES

4.6.1 General description and operational activities

The depot can be considered as the heart of a tramway system. It is the starting point of all trams from which they commence their transport services for their passengers. In general, depots are spacious areas which accommodate the trains when they are not in timetable service. Maintenance (light or heavy) also takes place in the same area. This includes small-scale repairs, sanding and cleaning.

The establishment of a new tramway depot is a tough procedure, since it takes place in an urban area with all the naturally ensuing problems. The main problem is finding a sufficiently large and available site; such large sites are rarely available inside the urban environment and are usually very expensive. Moreover, the selection of the location for the depot in an urban area almost always creates tensions and protests from neighboring residents.

The location and design of the depot significantly affects the overall operational cost of the tramway system. The depot should ideally be located as close to the tramway network as possible, in order to minimise the dead vehicle kilometers. Furthermore, all of the

Table 4.9 Facilities at a new tramway depot

Parking area/yard	Administration offices
Maintenance hall/workshop	Welfare facilities
Vehicle cleaning area	Waste storage
Warehouse (storage) area	Car parking space for employees and visitors
Painting workshop	

involved installations and facilities must be designed optimally, since any wrong estimation can increase the time and cost of activities performed, thereby increasing the total operational cost (Tramstore21, 2012a).

Table 4.9 presents the facilities of a tramway depot. Painting workshop can be caracterised as an optional facility.

The main design, constructional and operational characteristics of the essential facilities are presented in the following (Tramstore21, 2012a,b,c,d,e,f; Verband Deutscher Verkehrsunternehmen 823, 2001).

4.6.1.1 Parking area/yard

During the design of this area, the primary objective is to achieve the maximum tram parking capacity and a smooth flow of trains. The length of parking tracks depends on the number of trams which will park at each track, as well as on the tram's length. The lateral distance between parking tracks should be sufficient in providing a corridor of about 1.50 m between the sides of two parallel parked trams, in order to allow access by drivers, maintenance and cleaning staff. Thus, the size of the parking yard is the product of track length and track width, which depends on the number of tracks and their in-between distance.

Regarding the sheltering of the train parking area, there are three alternatives:

- Outdoor area
- Sheltered area (which features a roof with or without side walls)
- Indoor area (features a roof, side wallsand a front/end wall and access doors)

4.6.1.2 Maintenance hall/workshop

This facility includes workshops for heavy maintenance, light maintenance, bogie maintenance, vehicle cleaning area, sanding plant, electronic systems unit, track maintenance (rails and catenaries), as well as facilities for the auxiliary equipment.

The length of each track depends on the length of the vehicles; normally, a maintenance track should be sufficiently long enough to service at least one tram.

The number of tracks depends on the number of trains that are served in the particular depot, on the multitude and type of activities performed within the light maintenance workshop and on the overall configuration, layout and utilisation of the available space, maintenance-wise. In general, the maintenance facility area should be able to accept approximately 10% of the total number of trams normally served at the particular depot (Tramstore21, 2012a,b,e).

The lateral distance between two adjacent tram maintenance tracks should be sufficient in providing a corridor of about 3.5 m between the sides of two neighbouring trams. Within this space, the maintenance staff may move, place the required mechanical equipment, and perform all necessary activities.

4.6.1.3 Vehicle cleaning/washing area

For the washing of trains in most depots, either the 'drive-through system' or the 'gantry system' is used (Tramstore21, 2012c,f).

Regarding the positioning of these systems inside the depot area, it is considered preferable to locate them along the route section which the train follows from the moment it enters the depot till it reaches the parking area. With this configuration, 'dead' mileage can be avoided, and the vehicle may also enter the maintenance halls if required, in a state facilitating inspection by the maintenance personnel. Furthermore, many depots locate their sand silos between the entrance and the parking yard, namely before the cleaning area (Tramstore21, 2012c,f).

4.6.2 Classification of tramway depots

Tramway depots are classified as follows:

- According to the means of transportation that they serve:
  - *Exclusively for tram use*: Only tramway vehicles are served.
  - *Mixed use*: Besides tram vehicles, other mass urban transit means, such as buses and trolleys, are also served.
- According to the activities performed within their area:
  - *Fully operating*: All required activities are performed in the depot (see Table 4.9).
  - *Limited operating*: A limited number of activities are performed. This may occur in two cases: in the first one, some activities are outsourced to third parties; in the latter case, the tramway system includes more than one depot and the required activities are shared among them.
- According to the depot's location within the network:
  - *Central*: When the depot is located in the centre of the network. This 'gravitational' location is preferred when the network follows a radial-shaped development.
  - *Terminal*: When the depot is located at either end of the network. This position is preferred when the network follows a linear-shaped development.
- According to the size of the ground plan area:
  - *Very small*: Serving up to 25 tramways.
  - *Small*: Serving 25–35 tramways.
  - *Medium*: Serving 35–65 tramways.
  - *Large*: Serving more than 65 tramways.
- According to its accessibility from the main track:
  - Through a junction.
  - At the end of the main track as an extension.

4.6.3 Main design principles and selection of a ground plan area

The designer of a tramway system must be aware of the required area of the site during the early stages of the study so as to make an initial estimate of not only the cost of the tramway depot, but also the cost of the whole project. On the other hand, the operator needs to know in advance the required area of the site in order to proceed with their search and the procedures that will be required for its acquisition as quickly as possible (e.g., expropriation).

Currently, there are no regulated specifications for the design of a tramway depot. The literature references (Tramstore21, 2012a) and (Verband Deutscher Verkehrsunternehmen 823, 2001), provide the basic design, construction and operation principles without correlation with the train fleet (number of vehicles, train length). In this context, the design choices that are made and the final area of the tramway depot which they satisfy are governed by the initiative of the designers and the recommendations of the system operators. The cost of implementing a tramway depot is very high, and is around 20% of the infrastructure cost of a tramway system. The oversizing of the tramway depot increases the cost of the project significantly, while its undersizing results in problems in the system's operation.

The required area size of a tramway depot's ground plan depends on the following parameters:

- The fleet to be served (number of vehicles)
- The dimensions of the trains (length and width)
- The minimum allowed horizontal curve radius in the track alignment, and therefore, the geometric elements of switches and crossings
- The layout of parking and maintenance tracks for the trains
- The area size of main buildings and facilities
- The maintenance policy applied by the operator

In the initial fleet demand design, potential expansions should also be taken into account, regarding both the number of trams and their length, even mid-term.

It is recommended that the various facilities of the depot are interconnected. Additionally, the installation of a ring track (loop line), which circles around the area and is accessible from several points, is desirable. This allows trams to enter or exit the various facilities, such as the parking yard or the maintenance hall, without crossing through and occupying other areas (Tramstore21, 2012a; Verband Deutscher Verkehrsunternehmen 823, 2001). In general, the following routes should be possible without performing any maneuvers:

- Entry → Parking Yard → Exit
- Entry → Inspection and Cleaning → Parking Yard
- Parking Yard → Inspection and Cleaning → Parking Yard
- Entry → Maintenance hall → Parking Yard
- Parking Yard → Maintenance hall → Parking Yard

The warehouse (storage areas) should be located within or next to the maintenance workshop in order to reduce transfer times of the various materials. Administration offices, welfare facilities and the parking area for cars and motorcycles should be constructed at places where car and pedestrian movement do not conflict with operational tracks and tram movements.

Relevant literature (Chatziparaskeva et al., 2015) proposes a methodology which allows the estimation of the minimum required area of the ground plan of the various installations of a tramway depot and its total area, in relation to the fleet, the length of the trains, the number and type of activities performed.

The whole approach is performed with the aid of two 'tools':

- Statistical data from existing tramway depots
- Design simulation of the required facilities and integration into an overall layout/plan

The paper concludes with

- The formulation of simple mathematical expressions for calculating the useful area of individual facilities.
- The export of tables, which show the total area of the tramway depot.
- The configuration of the typical ground plans of the tramway depot sites which assign these findings for fleets between 15 and 80 vehicles, and train lengths of 30, 35, and 40 m.

Table 4.10 Example results from the design simulation – Estimation of the required area E_d of the tramway depot's ground plan for a fleet of 15–80 vehicles and for vehicle lengths of 30, 35 and 40 m

	Total area E_d (m^2)	*Total area E_d (m^2)*	*Total area E_d (m^2)*
Fleet	*Ratio x,y* *Tram length 30 m*	*Ratio x,y* *Tram length 35 m*	*Ratio x,y* *Tram length 40 m*
15	30,264 x = 1.4y	32,604 x = 1.4y	34,944 x = 1.4y
20	31,040 x = 1.1y	33,440 x = 1.1y	35,840 x = 1.40y
33	35,308 x = 1.7y	38,038 x = 1.5y	40,768 x = 1.3y
45	41,850 x = 1.1y	45,570 x = 1.2y	49,290 x = 1.2y
52	43,650 x = 1.6y	47.530 x = 1,6y	51,410 x = 1.7y
67	47,700 x = 1.6y	51.940 x = 1,6y	56,180 x = 1.5y
80	51,750 x = 0.9y	55,860 x = 1.7y	60,420 x = 1.6y

Table 4.10 provides example results of the design simulation of tramway depots for a fleet of 15–80 trains, and for vehicle lengths of 30, 35 and 40 m.

Figure 4.27 illustrates the variation of the tramway depot's total ground plan area in relation to the fleet, for three different tram lengths.

Figure 4.28 illustrates the variation of the tramway depot's total ground plan area in relation to the vehicle's length, for different fleet size.

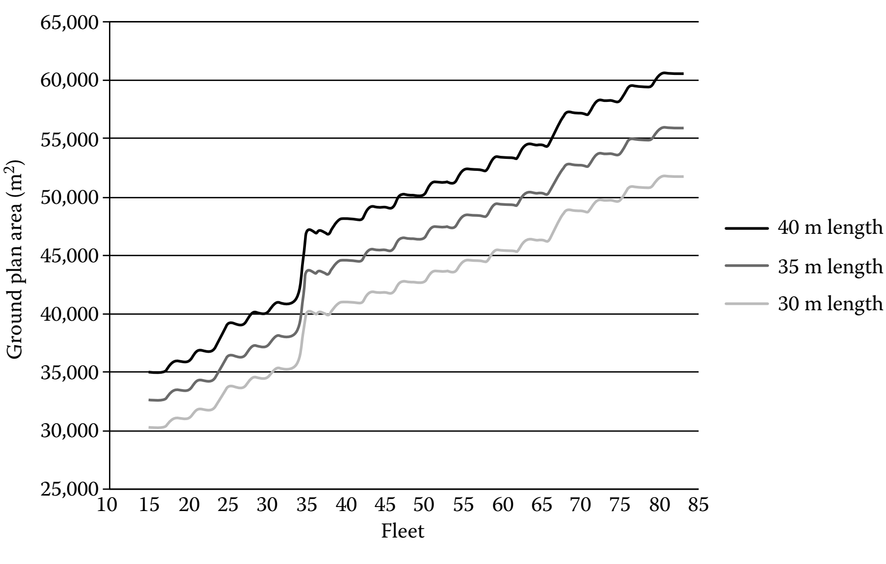

Figure 4.27 Variation in the total required ground plan area of the tramway depot in relation to the fleet, for different tram lengths. (Adapted from Chatziparaskeva, M., Christogiannis, E., Kidikoudis, C. and Pyrgidis, C. 2015, Estimation of required ground plan area for a tram depot, *Proc IMechE Part F: J Rail and Rapid Transit* 1–15, IMechE 2015 DOI: 10.1177/0954409715570714.)

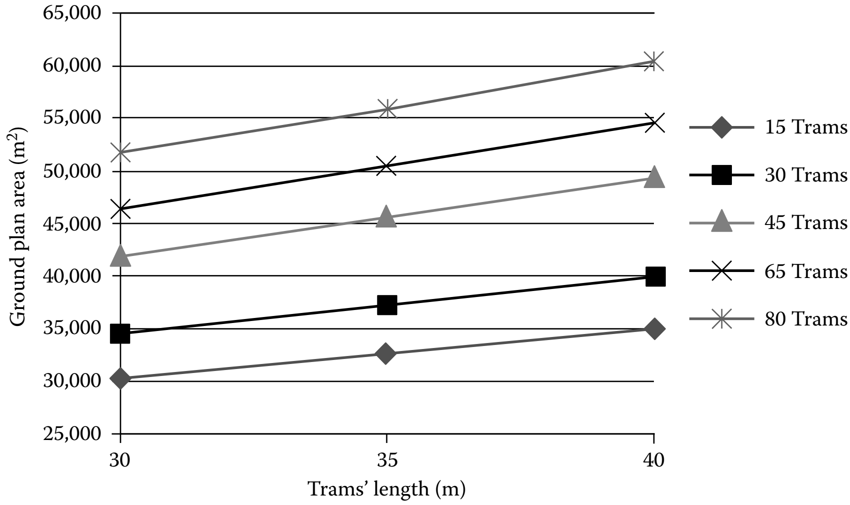

Figure 4.28 Variation in the total required ground plan area of the tramway depot in relation to the train length for different fleet sizes. (Adapted from Chatziparaskeva, M., Christogiannis, E., Kidikoudis, C. and Pyrgidis, C. 2015, Estimation of required ground plan area for a tram depot, *Proc IMechE Part F: J Rail and Rapid Transit* 1–15, IMechE 2015 DOI: 10.1177/0954409715570714.)

By studying Figures 4.27, 4.28 and Table 4.10, the following conclusions can be reached:

- The construction of depots for 15 vehicles (very small depots) requires an area of 30–35 acres.
- The construction of depots for 45 vehicles (medium depots) requires an area of 42–50 acres.
- The construction of depots for 80 vehicles (large depots) requires an area of 52–60 acres.

Figure 4.29 illustrates an example of a layout of the configuration of a tramway depot aiming to serve a fleet of 45 vehicles with a 35 vehicle length, as a result of a design simulation.

4.7 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

Generally, the tramway system is selected as a means of transport

- When there is relatively low demand for travel (≤10,000–15,000 passengers/h/direction).
- When it is sought to regenerate/upgrade an area and, generally, when it is desired to maintain the activities of an area.
- For cities facing a specific air pollution problem.
- When there is high demand for travel (>10,000 passengers/h/direction) and the subsoil or the lack of funding prevents the implementation of an underground solution.

The tram is the only urban public transport mode which results in the active removal of private cars from the areas through which it passes, with concurrent satisfaction of the demand. This is mainly achieved by integrating the tramway system accross existing roads, thereby substantially reducing the street parking spaces for private cars.

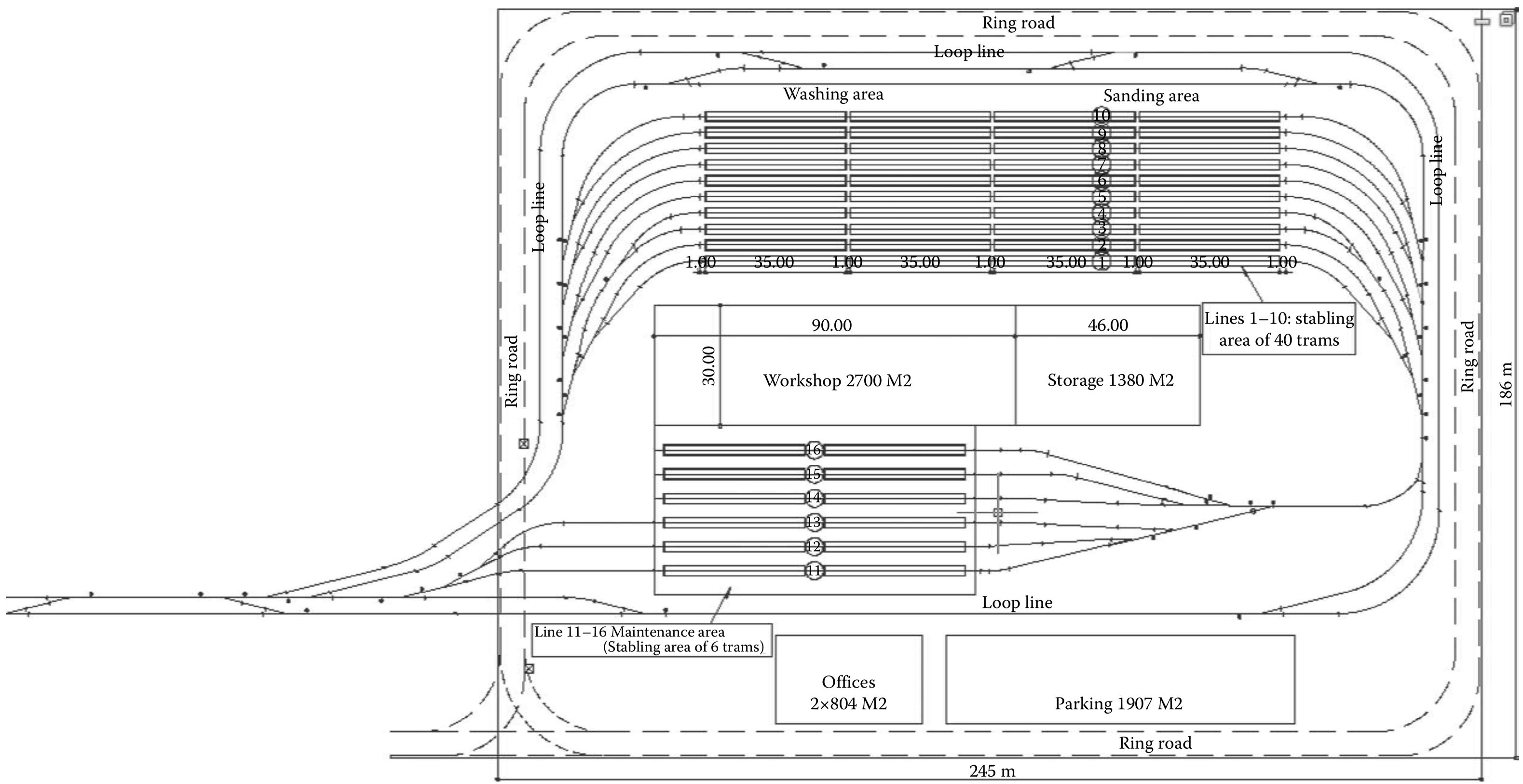

Figure 4.29 Example layout of the configuration of a tramway depot for 45 vehicles with a length of 35 m. (Adapted from Chatziparaskeva, M., Christogiannis, E., Kidikoudis, C. and Pyrgidis, C. 2015, Estimation of required ground plan area for a tram depot, *Proc IMechE Part F: J Rail and Rapid Transit* 1–15, IMechE 2015 DOI: 10.1177/0954409715570714.)

Implementing a tramway system is politically the easiest way to 'claim' a dedicated lane for public transport in the urban space. Also, it is easier to make decisions regarding pedestrianisations in cities which feature a tramway system.

4.8 APPLICABILITY VERIFICATION OF ALTERNATIVE ALIGNMENTS

Figure 4.30 presents the successive stages of a tramway project study. More specifically, in the *first stage*, alternative alignments are identified and preselected, according to the

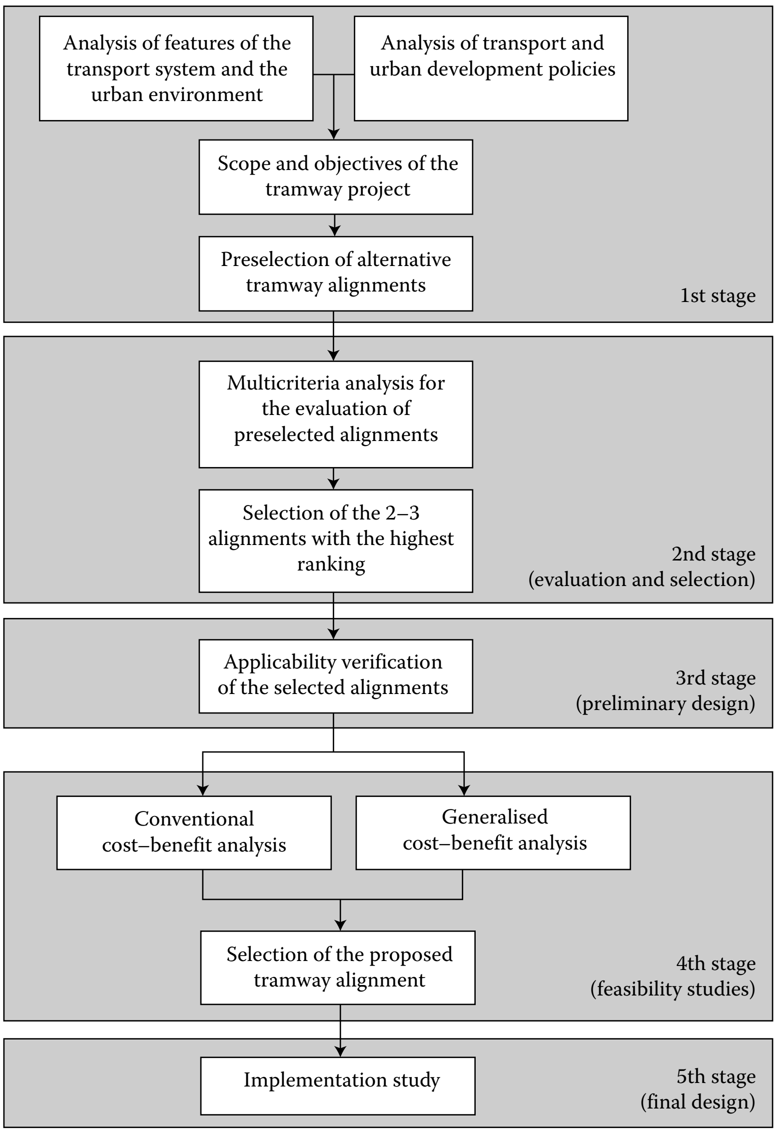

Figure 4.30 Stages of a tramway system design study. (Adapted from Pyrgidis, C., Papaioannou, P., Gavanas, N. and Politis, I. 2015, A methodology for the evaluation of alternative tramway alignments in the early stage of the feasibility study and application to the city of Thessaloniki, Greece, *Rail Engineering International*, 2, 11–16.)

objectives and prerequisites set by the contracting authority, such as the starting and ending points of the alignment, the areas to be served, the accomplishment of the priorities for future sustainable mobility etc. The number of these preselected alignments is usually high (more than five alternative alignments).

At the *second stage*, the aforementioned alternative alignments are evaluated, and those with the highest potential (usually 2 or 3 of them) qualify for the subsequent stages of study. The attained evaluation process can be performed by means of a multicriteria analysis (Pyrgidis et al., 2015).

The third stage focuses on the applicability verification of the alignments that received the highest score during the multicriteria evaluation. The applicability verification is a special stage in the implementation studies of urban and suburban rail networks. It has the same weight and importance as a preliminary track alignment study and a prefeasibility study. It formulates feasible solutions while at the same time it produces the constructional, operational and financial data of the system which are essential for the final studies (Pyrgidis, 2003).

In particular, it is essential that the 2–3 selected alignments satisfy the following conditions:

- Is the geometrical insertion of the alignments feasible?
- Do they ensure an adequate level of service to the users (low travel time, frequent service)?
- Can their possible negative impacts on the other transport modes be overcome in an effective and low-cost way?
- Do they create negative environmental impacts on the area of influence, which may be prohibitive for the implementation of the tramway project?
- Is there space availability for the construction and efficient operation of rolling stock parking, maintenance and repair facilities (depot)?
- Does the preliminary estimation of the cost of the tramway project in accordance with the international practice (average cost per track kilometer)?

The satisfaction of the above conditions prerequires, for each alternative alignment, the applicability verifications illustrated in Figure 4.31 which must be satisfied simultaneously.

These applicability verifications may introduce minor or major modifications to the initial design.

At the *fourth stage,* the feasibility study of the qualified alignments is conducted, using two approaches: (a) the conventional cost–benefit analysis which evaluates the net financial benefits expected to derive in a predefined time horizon and (b) the generalised cost–benefit analysis which evaluates the additional monetary values of the expected socioeconomic, environmental and land use benefits and costs.

The *fifth stage* pertains to all the other studies required for the final implementation of a tramway project.

4.8.1 Verification of track alignment and geometric integration

During this verification, the designers, having shortlisted a potential alignment of the tramway network, check whether

a. The alignment is acceptable both for the horizontal and the vertical.
b. Along the transit zone, there is adequate space for the integration of the tramway infrastructure and for the movement of pedestrians and road vehicles, with a level of service that is considered acceptable.

Figure 4.32 presents a logical diagram of the steps that designers follow during the process of the applicability verification of the track alignment and geometric integration.

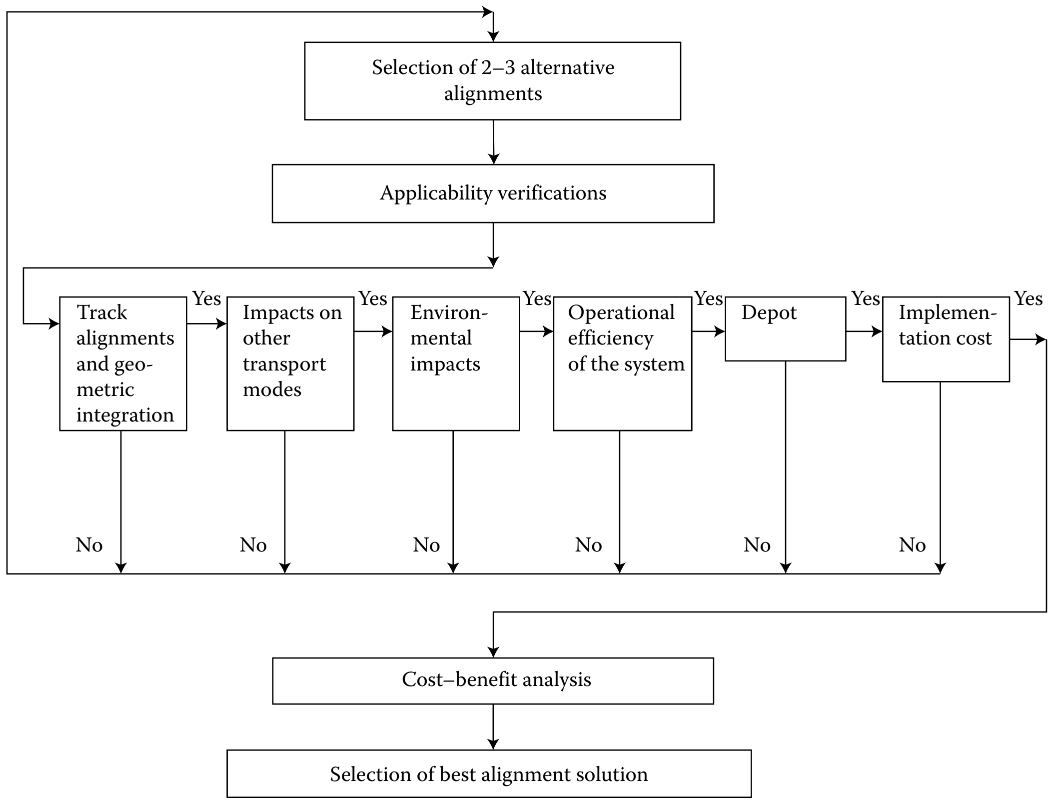

Figure 4.31 Logical diagram of alternative tramway alignments acceptance. Applicability verifications.

As can be seen from Figure 4.32, while carrying out this verification, two individual verifications are conducted in chronological order, namely

1. *Track alignment verification:* At this stage, it is checked whether the track alignment allows, horizontally and vertically, the running of trams. More specifically, the radius of curvature in the horizontal alignment R_c, the gradient i in the vertical alignment, and the height clearance h under the civil engineering structures are checked. For the verification of the above parameters, their calculation along the entire network and the comparison of their values to the minimum/maximum permitted values are required. If the verification is satisfied for all parameters, the designers proceed to the verification of the geometric integration.
2. *Geometric integration verification:* At this stage, and having
   - Preselected the category of tramway corridor for each road artery (classes A, B, C, D, E)
   - Preselected the final integration type of the tramway tracks
   - Preselected the vehicle height
   - Selected the integration type of tramway stops
   - Selected the power supply system and, in the case of electrification, the type of integration of the overhead wires
   - Selected the width and length of the vehicle

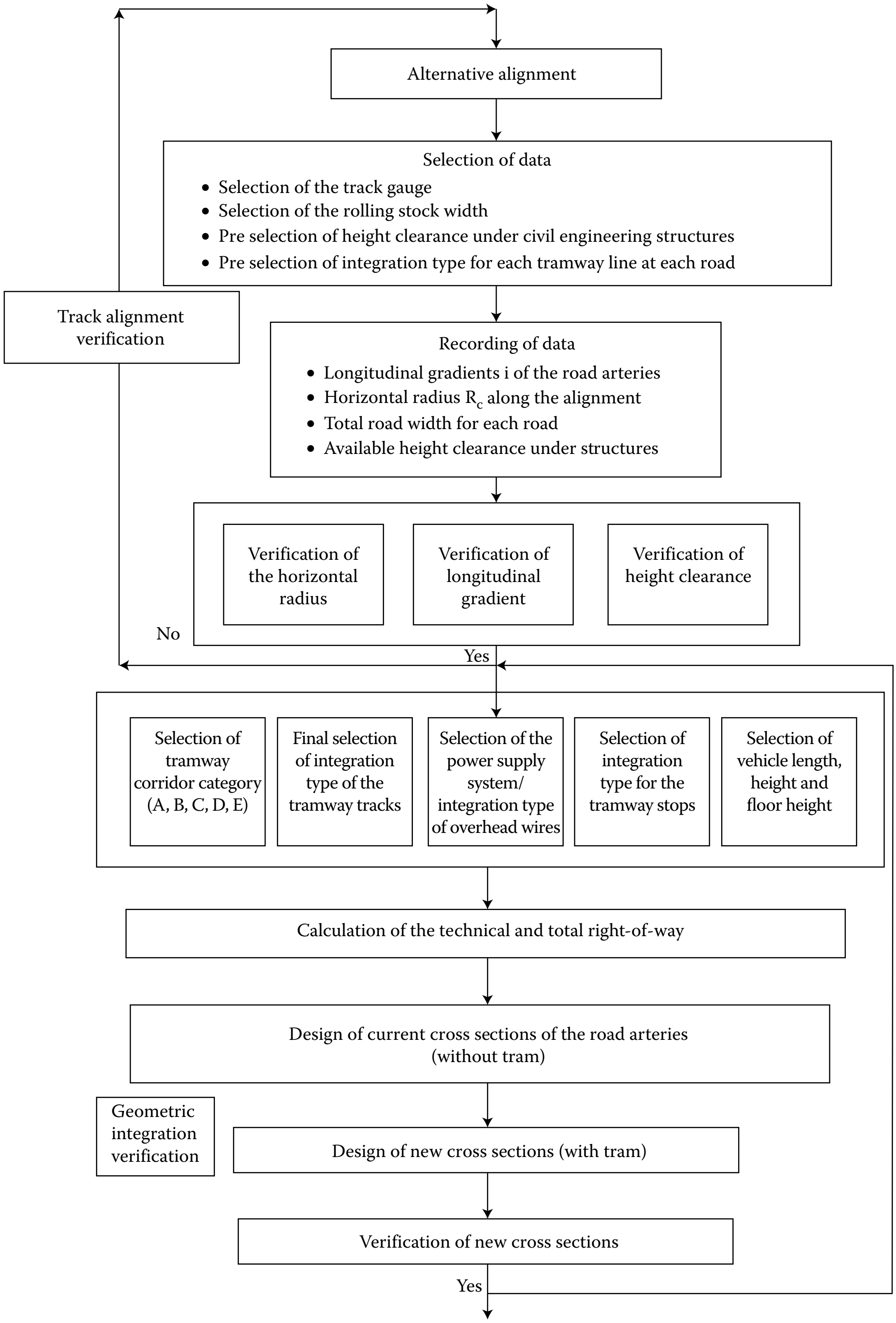

Figure 4.32 Verification of the design and geometric integration of alternative tramway alignment.

The technical and total tramway infrastructure right-of-way are calculated

- For straight paths as well as for the areas of stops with the aid of mathematical expressions (4.1 through 4.11)
- For turns, with the aid of design simulation

In order to enable the geometric integration of tram lines in the straight paths as well as in the areas of stops, the total right-of-way should be less than the available road width.

In order to enable the geometric integration of tram lines in curves, even with the smallest allowable radius of 25 m, the two intersecting roads should have the available width (Chatziparaskeva and Pyrgidis, 2015).

The current cross sections are then designed at strategic locations of the alignment, and the condition of the road is recorded concerning the circulation of road vehicles and pedestrians before the integration of the tramway system.

Finally, the new cross sections are designed at the above mentioned locations, taking into account the integration of the tramway system.

If the new situation is not acceptable (e.g., inadequate width of sidewalks, inadequate width or inadequate number of traffic lanes), then intervention is required for the modification of one or more of the above initial selections in order to achieve an acceptable geometric integration. In the case where the new situation, arising after the integration of the tramway system, is acceptable, then the designers proceed to conducting the next applicability verification for the alignment that is under examination.

Figure 4.33 and its attached table illustrate an example of the current layout and the new layout for a road before and after the integration of a tramway system (placement of a single tramway track at two opposite sides of the roadway).

Figure 4.33 Example of the current layout and the new layout for a road before and after the integration of a tramway system-Queen Olga Street, Thessaloniki, Greece. (Photo: N. Regka.)

Road width	*Sidewalk width*	*Current layout*	*Total right-of-way by the tram*	*Remaining road width*	*New layout*
14.30 m	3.16 + 3.16 m	4 × 3.75 m ↓	6.40 m	7.90 m	1TT + 2 × 3.50 m ↓ + 1TT

where

TT: single tramway track

↓: direction of road traffic.

4.8.2 Verification of impact on other transport modes

The impacts that can arise from the integration of a tramway system in an urban area mainly involve

- The traffic of road vehicles
- Parking (reduction of on-street parking spaces)
- Service of roadside land uses
- Pedestrian movement (decrease of the sidewalk width, crossings)
- The operation of public transport modes (PT)

Figure 4.34 illustrates a logical diagram which ensures that the possible negative impacts on other transportation modes arising from the integration and operation of the tramway system in regard to urban area can be removed or limited to acceptable levels.

The assessment of the new traffic conditions for the road network is usually conducted with the assessment of the trip origin–destination matrice, and its assignment on the network that is being examined with the aid of an appropriate mathematical simulation model. At the same time, it is necessary to record the removed (and therefore eliminated) parking spaces, and to explore how and where they can be recreated.

4.8.2.1 Roadside land uses

The impact from the integration of the alignment that is being examined in relation to the service of roadside land uses is considered

- Basing on the service requirements of specific land use categories and, particularly, the service of commercial uses by feeding vehicles and
- Basing on the access to specific uses (such as hospitals, sport facilities, exhibition and convention centres and educational institutions)

4.8.2.2 Pedestrians

The pedestrian network in the study area is favoured by the integration of a tramway system for three main reasons: (a) roads are rearranged in terms of traffic and parking improvements which results in the improvement of pedestrian safety and of the mobility conditions for vulnerable users, (b) the pedestrian areas surrounding the tram are environmentally upgraded and (c) the crossing of streets at unprotected locations is averted due to the existence of the tramway track superstructure (protected crossings are foreseen). Furthermore, the combined use of walking and tram enhances pedestrian flows in areas

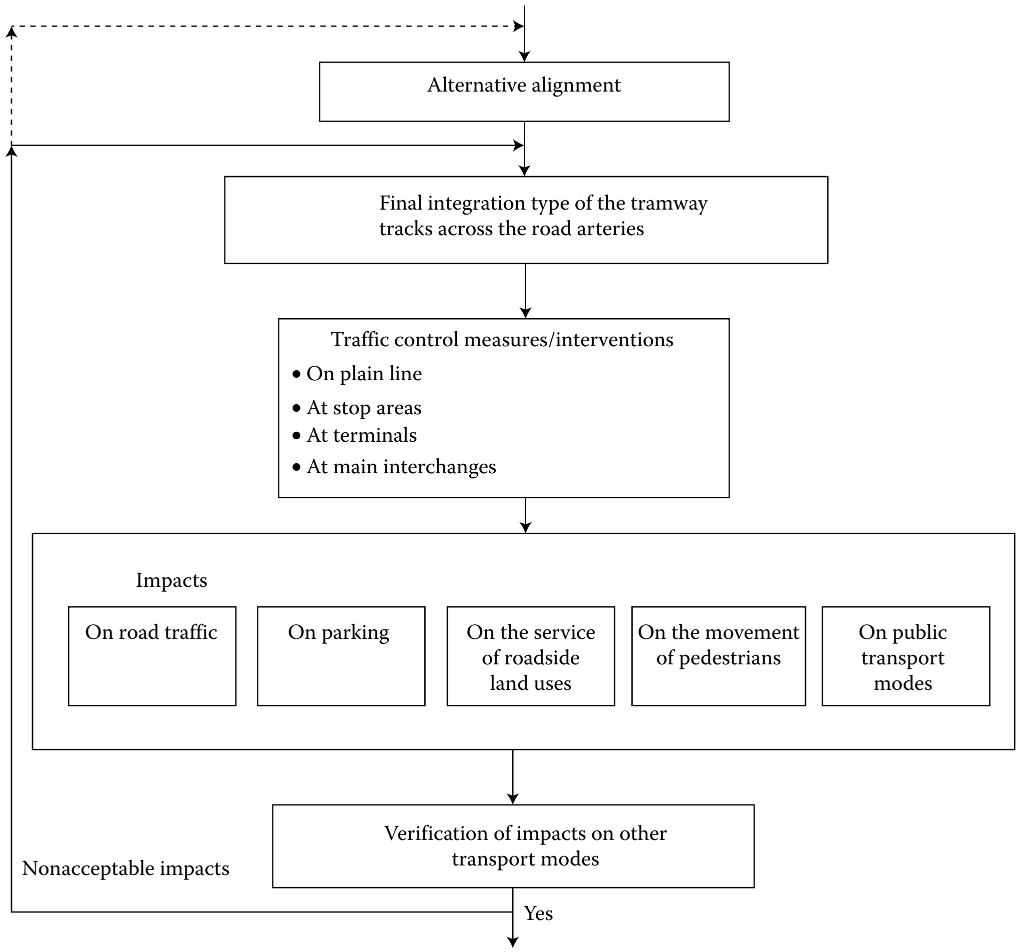

Figure 4.34 Applicability verification concerning the impacts resulting from the integration and operation of a tramway system on other transport modes.

of interest, such as the historic and commercial centre of the city, the services and the recreation areas.

The operation of the tram generally does not impede cycling; on the contrary, the replacement of part of the polluting road traffic by a tramway system which is environmentally friendly, upgrades the conditions for cyclists and promotes cycling.

4.8.2.3 Operation of other public transport modes

The integration of a tramway system in an urban area requires a study for the restructuring of the timetables and scheduled services of bus lines. This parameter is of great importance as often different operators with conflicting interests are involved.

4.8.2.4 Road traffic

The impacts on the road width that is available for the movement of motor vehicles along the roads on which the tramway system is integrated can be distinguished as

- Limited impacts, that is, impacts which are not expected to cause significant changes in the existing operation of the roads and the service of road traffic
- Important impacts which affect the existing operation of roads

Another significant impact from the integration of a tramway system on the road traffic concerns the facilitating of certain turning movements and the direct access of road vehicles (passenger and feeding) to the adjacent land uses. In any case, all of the aforementioned impacts can be encountered by integrating the tramway system on a tramway corridor class E and, more importantly, by using appropriate signalling at intersections, and by ensuring the installation of adequate guidance and warning signage, both vertical and horizontal.

4.8.3 Verification of environmental impacts

The effects of a surface railway transport mode on the environment cannot be considered as negative. However, as such systems pass through densely populated areas, it is likely to increase the noise level, while at the same time their operation requires equipment (overhead wires, rails, masts, etc.) that can cause visual or general aesthetic problems. At the same time, their positive impacts on the urban environment and, generally, on the upgrading of the urban environment (e.g., reduction of air pollution, rehabilitation and regeneration of certain areas) cannot be ignored.

The main challenge during the verification of the environmental applicability is whether the designer considers these effects negligible compared to the functionality of the network, or sufficient enough to require a radical restructuring of the study and, therefore, the construction of the project. In the second case, an assessment of the problems and the possible countermeasures should be made in order for the integration of the tramway system to be smooth.

In summary, all of the above are illustrated within the logical diagram of Figure 4.35.

4.8.3.1 Noise pollution

Noise is referred to as the main polluting factor of a tramway system. Noise produced by a tramway system can be due to (a) the overhead power supply (arc noise) and (b) the train movement (rolling noise and vibrations).

Rolling noise is the most significant disturbance, as it occurs on the wheel-rail contact surface and is due to the lateral and longitudinal creep forces, the guidance forces exerted on the contact surface of the inner rail and the wheel flange and the vertical dynamic loads, which generate the vibration. If the rails are positioned correctly with the appropriate provisions and vibration damping mechanisms, the passage of a tram causes less noise than any other transport mode, while vibration is limited to a minimum.

The impact of this noise is critical in case the tramway passes close to specific land uses, such as health facilities.

The estimation of the impact of noise pollution should be considered in conjunction with the limitation of noise due to the reduction of motorised traffic and the regulation of the traffic flow.

For the noise impact at critical areas, a special study is required as part of the design study of the alignment. When these critical facilities cannot function appropriately the interventions relating to noise reduction may involve

- Placement of the track on the side of the road that is opposite to the facility thereby increasing the distance between the facility and the noise source.

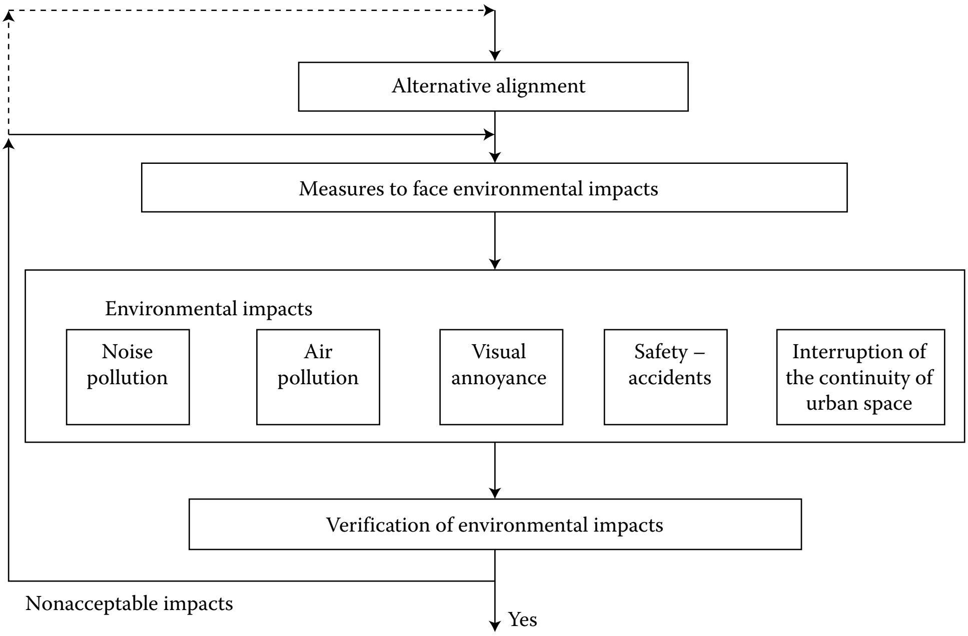

Figure 4.35 Applicability verification concerning the environmental impacts resulting from the integration and operation of the tramway system.

- Deployment of a floating slab of sufficient length (100 m) before and after the location of the facility.
- Use of noise barriers for parts of the alignment outside the main urban space.
- Regular and appropriate maintenance of the track and the vehicles.
- Resilient wheels.

4.8.3.2 Visual annoyance

The main parameter of visual annoyance related to the urban surface tramway system is the overhead catenaries. The aesthetic nuisance is important both in developed areas with a high population density and visitors, and in less developed areas, as they limit their attractiveness.

The aesthetic nuisance should be counterbalanced by the improvement of the urban environment due to the reduction of the number of road vehicles and, in particular, the reduction of congestion in central areas. In addition, in order to reduce the aesthetic degradation resulting from the use of overhead catenaries, an appropriate design should be applied which may reduce the exposure of any additional equipment, and may minimise the overhead infrastructure with the aid of planting and other aesthetic means.

Nowadays, free catenary power supply systems are in operation. These systems (see Chapter 20) provide a technical solution to this problem, especially when the tram passes largely through the historic centres of major cities.

4.8.3.3 Impact on the urban space

The reorganisation of road infrastructure and traffic through the development and operation of tramway systems and the relief of congestion enables redesigning of central areas, upgrading of the urban environment in deprived areas, and increasing the attractiveness in new urban areas.

4.8.3.4 *Impact on safety*

In the case of a nonsegregated tramway corridor, the coexistence of tramway traffic with road traffic and pedestrians and other road users may result in accidents such as pedestrian entrainment or collisions with private cars, at a much higher frequency than the respective frequency of such accidents in the suburban or interurban railway. Cyclists comprise a particularly vulnerable user group, followed by pedestrians and motorbike riders. The problem is exacerbated as the death rate for the above user groups, when they are involved in an accident with a tram, is greater than the respective rate for accidents involving private cars.

Furthermore, the service of a large number of tramway users and their frequent movement to and from the stops/platforms, makes it more likely that they become involved in accidents such as falling off the platforms, getting trapped between two trams or between a tram and a road vehicle, entrainment by road vehicle in the course of their approach to the station/platform or during their departure from it, etc.

Finally, level crossings are a crucial point of tramway network, both in terms of safety and in terms of operation. This is due to the fact that they constitute a conflict point with road traffic and pedestrians.

It should be noted that most accidents involving a tram take place in the first months of its operation, as private car drivers and pedestrians are not used to the integration of the tramway system within the city's transportation network. Basing on the international experience, despite the additional accidents caused by the tram, the total number of various transport modes (private cars, trams, bicycles and motorbikes) involved is less than before the integration of the tramway system, as a result from the reduction of vehicle kilometers run by road transport (private cars, bicycles and motorbikes).

4.8.3.5 *Impact during construction*

During the construction, various problems can be caused in the area; however, compared to the problems caused during the construction of a metro system, those problems are of a much smaller scale.

4.8.4 Applicability verification of operational efficiency

While carrying out this verification, the designers consider whether the alignment that is being examined is operational, emphasising on the commercial speed of the tramway vehicles and the passenger transport volumes.

Therefore, as shown in Figure 4.36, the verification of the system's operational efficiency includes three individual verifications, which are carried out in the following order:

1. Verification of the commercial speed
2. Verification of the passenger transport volumes
3. Verification of operating cost

4.8.4.1 *Verification of commercial speed*

While carrying out this verification, it is investigated whether the commercial speed V_c of the trams is considered satisfactory by the users.

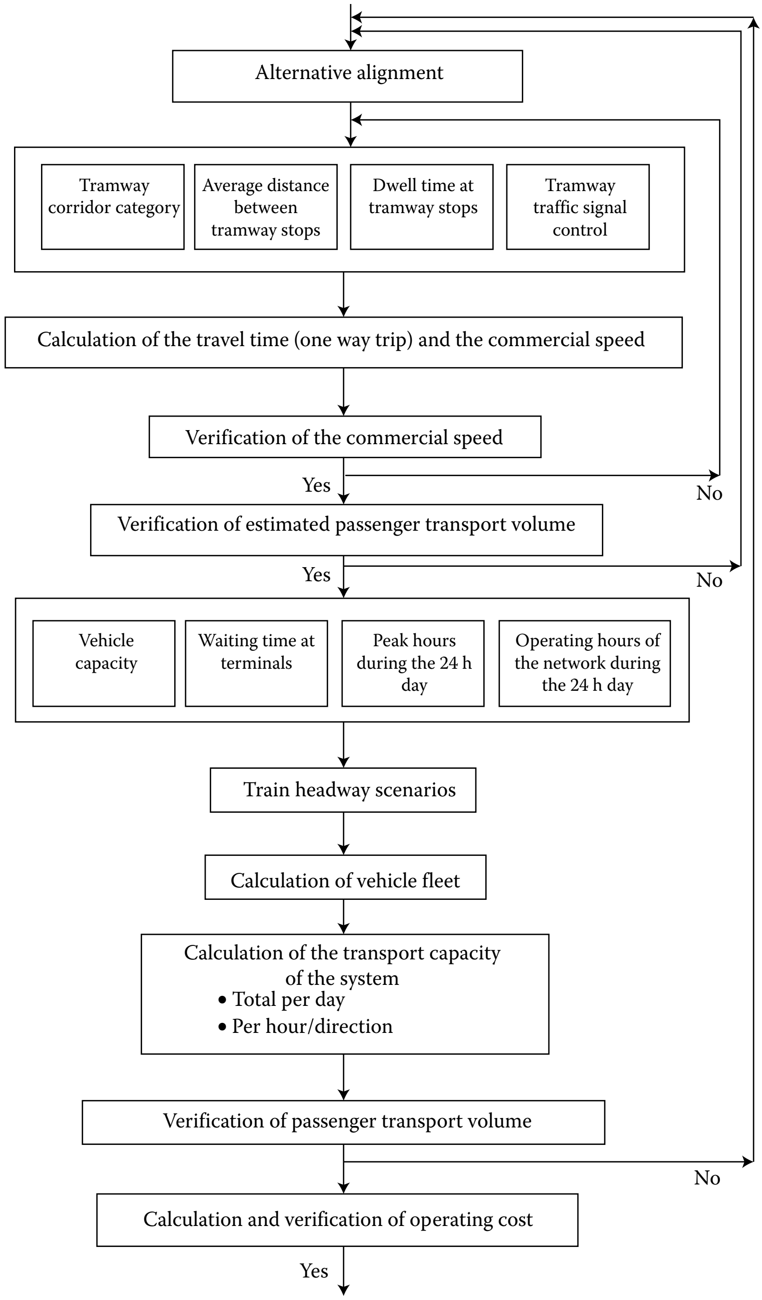

Figure 4.36 Verification of operational efficiency.

This takes onto consideration the category of tramway corridor along every road artery and the total length of each corridor category and by preselecting:

- The intersections with roads where the tram will have priority at traffic signals
- An average distance between successive stops equal to 500 m
- An average waiting time at each stop equal to 20 sec

the commercial speed V_c of the trams, and the travel time t, can be calculated (Bieber, 1986).

$$t = \frac{S_A}{V_{cA}} + \frac{S_B}{V_{cB}} + \frac{S_C}{V_{cC}} + \frac{S_D}{V_{cD}} + \frac{S_E}{V_{cE}} \tag{4.12}$$

$$V_c = \frac{S}{t} \tag{4.13}$$

where

V_c: Commercial speed
S: Total route length
t: Total travel time
S_A, S_B, S_C, S_D, S_E: Tramway corridor length for corridor categories A, B, C, D, E, respectively
V_{cA}, V_{cB}, V_{cC}, V_{cD}, V_{cE}: Commercial speed of tramways running on corridor categories A, B, C, D, E, respectively (Table 4.1).

If the resulting commercial speed is not within acceptable limits (typically 18–25 km/h) or a travel time which was initially set as a target is not met, the verification is repeated after modifying one or more of the above options (e.g., priority to tram at all traffic signals, change of the tramway corridor category). For example if priority is given to the tram at all signalised intersections, and if it is considered that this causes an increase in the commercial speed by 25%, the mathematical equation applies where $V_{cB} = 25$ km/h and $V_{cD} = 22.5$ km/h.

4.8.4.2 Verification of passenger transport volume

At this stage, two verifications are performed on the passenger transport volume, in the following order:

1. *Verification of the estimated passenger transport volume (P_d):* The daily estimated passenger transport volume is compared with the typical values of passenger transport volumes of similar passenger tramway systems as they are referred to in the international literature. The value of the estimated passenger transport volume P_d has been produced with the aid of traffic forecasting models.

   The estimated passenger transport volume must meet the minimum volume requirements and document the need to investigate the feasibility of implementation of a tramway system in this area.
2. *Verification of the transport capacity of the system:* The daily passenger P'_d volume that can be carried by the tramway system is compared with the estimated P_d value.

   To calculate the capacity of the system, the values of the following parameters are initially selected:
   - The transport capacity C_v of tramway vehicles.
   - The waiting time t_{ts} of the trains at both terminals.
   - The peak hours within the 24 h day.
   - The duration of the network operating hours within the 24 h day.

   Considering various scenarios regarding the headway between trains, the following parameters can be calculated:

- The required fleet of vehicles (including spare vehicles).
- The ridership that the system is capable of accommodating (total daily and total annual number of passengers/h/direction).

If the ridership that can be accommodated by the system is less than the estimated ridership, then designers modify one of the above parameters in an attempt to reach an acceptable solution for the passenger transport volume.

EXAMPLE

Total expected passenger volume per day per direction

- $P_d = 25{,}000$ passengers (optimistic scenario)

Expected daily passenger volume per direction during peak hours

- $P_{dph} = 4{,}000$ passengers (optimistic scenario)

Route length (AB) $S = 10$ km
Total travel time $t_{AB} = 0.5$ h = 30 min (Equation 4.12)
Commercial speed $V_c = 20$ km/h (Equation 4.13)
Waiting time of trams at each terminal station $t_{ts} = 4$ min
Two-way route travel time (round trip + waiting time at the two terminal stations) $t_{ABA} = 2 \times 30 + 2 \times 4 = 68$ min
Train transport capacity $C_v = 200$ passengers (150 standing and 50 seated – density 4 passengers/m^2 – during off-peak hours)
Train transport capacity $C_{vph} = 275$ passengers (225 standing and 50 seated – density 6 passengers/m^2 – during peak hours)

The tram is considered operational from 05:30 to 00:30, that is, for a total of 19 h, while four of these operating hours are considered as peak hours.

Assuming a train headway for the operating hours during the off-peak period is equal to 10 min, a total of 68/10 = 6.8 = 7 'vehicles' are required to ensure the operation of the network. This figure should be increased by

- One replacement vehicle at the terminal or the tramway depot for the replacement of any vehicle that is damaged during operation.
- 12% (percentage of immobilised vehicles based on experience) of the estimated initial number of vehicles, namely one vehicle intended to replace any vehicle that is immobilised for repair or maintenance purposes at the tramway depot (Baumgartner, 2001).

Therefore the total required fleet of vehicles for the service of the line during off-peak hours is marginally equal to nine (9) vehicles.

As regards the transport volume, the tramway system can carry in total ((200 passengers × 60 min)/10 min) × 15 h = 18,000 passengers per direction during the 15 off-peak hours of network operation.

Considering a train headway that is equal to 7 min during peak hours, a total number of 13 vehicles are required.

As regards the transport volume, the tramway system can carry in total ((275 passengers × 60 min)/7 min) × 4 h = 9,428 passengers per direction during the 4 peak hours of network operation.

Therefore, considering a train headway that is equal to 10 min during the off-peak hours and 7 min during the peak hours, a total number of 13 vehicles are required for the smooth operation of the system. The system is able to carry

- 18,000 passengers per direction in total during the off-peak hours
- 9,428 passengers per direction in total during the peak hours (i.e. 2,357 passengers per hour)

This results to a total of 54,856 passengers during the operating hours of the tramway network.

Therefore, the estimated passenger volume in an optimistic scenario (25,000 × 2 = 50,000 passengers) is satisfied.

4.8.4.3 Verification of operating cost (K_{op})

While carrying out this individual verification, initially the operating cost of the network (cost of vehicles' circulation, cost of the power supply, track and rolling stock maintenance cost, cost for the operation and maintenance of fixed facilities, cost of infrastructure insurance, administration cost and other unforeseen costs) are calculated.

The resulting value of the cost is then divided by the estimated annual passenger volume, resulting in the operating cost per transported passenger, which is then compared with the average value that is usually met in practice internationally (this value is around €0.8).

4.8.5 Applicability verification of a tramway depot

The applicability verification of a tramway depot is one of the most important verifications, as the depot is probably the most important structural element of the tramway system. As illustrated in Figure 4.37, the applicability verification of the tramway depot includes four individual verifications which are performed simultaneously. These are

1. *Verification of the required and the available tramway depot ground plan area:* As part of this verification, initially the ground plan area that is required for the tramway depot, the lying of the tracks and other facilities, and the operation of the depot are calculated.

   For the estimation of the required ground plan area, the methodology that was described in paragraph 4.6.3 can be applied (Chatziparaskeva et al., 2015).

   The designers then compare the size of the area that is proposed for the location of the depot (available land) with the estimated required area.
2. *Verification of the distance of the tramway depot from the tramway network (verification of 'dead' mileage):* As part of this verification, the designers calculate the distance between the point of entry to the area of the depot and the nearest terminal in order to assess the 'dead' kilometers.

   The term 'dead' vehicle-kilometers describes the total vehicle-kilometers travelled by a vehicle without the production of any transport work. Therefore, the vehicle-kilometers to/from the depot are considered 'dead'. The nonproductive vehicle kilometers significantly affect the operating costs as their increase leads to an increase in energy consumption, the driving hours and rolling stock and track maintenance costs.

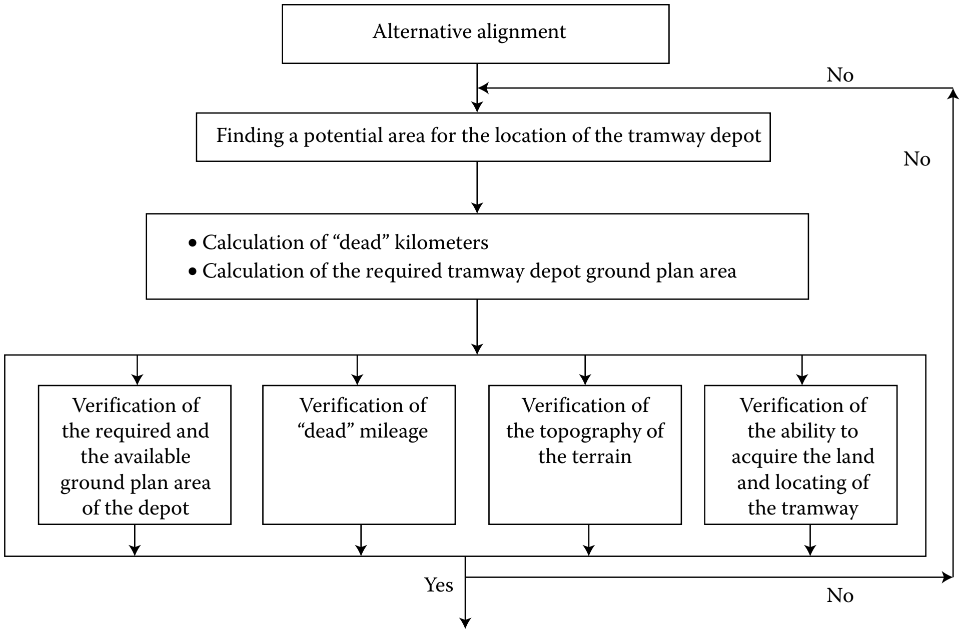

Figure 4.37 Applicability verification of a tramway depot.

The entrances/exits of the tramway depot should be located as close as feasible to the network of the main tram traffic lines. The maximum permissible distance between the entrance of the depot and the nearest terminal is 2 km.

3. *Verification of the topography of the terrain:* As part of this verification the longitudinal gradients of the potential construction area of the tramway depot are examined and compared with the maximum permissible value. The soil must feature a gentle slope in longitudinal profile, so as to facilitate the laying of the tracks with the permissible maximum longitudinal gradient. It should be noted that construction-wise, it is feasible to achieve the desired parameters, however, this can significantly increase the construction cost.
4. *Verification of the ability to acquire the land and locating of the tramway depot:* As part of this verification, the qualitative parameters are examined, contrary to the three aforementioned individual verifications which related to quantitative parameters. More specifically, the following are considered:
   - The possibility of obtaining an area for the construction of the tramway depot. Generally, areas that require the lowest cost of expropriation should be selected.
   - The compatibility with adjacent land uses. If the land is located in an area with incompatible land uses (residential, entertainment, health), other areas must be sought or environmental interventions should be applied, which may increase the construction cost.
   - The integration with the environment. The integration of the tramway depot within a 'sensitive' environment should be avoided. Generally, it should be possible to apply all necessary measures in order to minimise the environmental impacts at the catchment area of the tramway depot. Locating of the tramway depot at areas of archaeological interest should also be avoided.

4.8.6 Verification of implementation cost

The implementation cost comprises the construction cost of the infrastructure and the cost of acquiring the rolling stock.

The construction cost includes

- The cost of repairing the roads on which the tram is integrated
- The cost of relocating utility networks (gas, electricity, water, sewage)
- The construction cost of the subgrade
- The construction cost of the track superstructure
- The cost of equipment for tramway stops
- The cost of pedestrianisation in exclusive tramway corridors (where provided)
- The construction cost of civil engineering works
- The construction cost of depots
- The cost of installation of the electrification system, the signalling system and the telecommunications system
- The cost of construction of the necessary buildings for the system's operation
- The cost of studies, supervision and management of the project
- The cost of measures to face environmental impacts
- The expropriations cost

The cost of the project must be similar to the cost which is referred to in international practice. The average construction cost of a tramway line (infrastructure and rolling stock) is calculated at €20–25 M per track-km (2014 data).

4.9 HISTORICAL OVERVIEW AND PRESENT SITUATION

4.9.1 Historical overview

The evolution of the trams consists of five distinct periods. The period of the horse-drawn tram, the transition period from horse power to electric power, the period of development of electric trams, the period of the dismantling of trams and, finally, the period of the reintegration of trams in the urban transportation systems.

4.9.1.1 The first horse-drawn tram

The first passenger tram in the world was the Swansea and Mumbles Railway in Wales which commenced operation in 1807 as a horse-drawn tram. From 1877 to 1929 this tram was powered by steam.

The first tramway lines were laid in the United States, specifically in Baltimore, in 1830, in New York in 1832 (New York-Charlem tram line), and in New Orleans in 1834 (the oldest tram network with continuous operation worldwide). In Europe, the first tramway line was laid in France, near St. Etienne, in 1838.

In 1853, the first tram with grooved rails commenced operation in the Broadway Avenue of New York. These new tracks were soon available also in Europe and were invented by Alphonse Loubat.

The new transport mode was disseminated relatively fast, and by the end of the nineteenth century and the beginning of the twentieth century several big cities worldwide featured horse-drawn tram transport.

4.9.1.2 The transition period from the horse-drawn tram to electrification

Mechanical systems developed rapidly, beginning with the steam-powered systems in 1873 and continuing with the electric trams after 1881 when Siemens presented the first electric powered vehicle at the International Electricity Exhibition in Paris.

The steam-powered tram appeared in Paris in 1878. The first prototype of an electric tram was developed by the Russian engineer Fyodor Pirotsky, who converted a horse-drawn tram into an electric tram. His invention was trialed in St. Petersburg, Russia, in 1880. In 1881, Werner von Siemens opened its first electric tram line in the world at Lichterfelde near Berlin.

In 1883, Magnus Volk constructed an Electric Railway (Volk's Electric Railway) along the east coast in Brighton, England. This two kilometers line remains in service until today and is the world's oldest electric tram which is still functional.

The first major electrical system in Europe operated in Budapest since 1887.

Parallel advances took place during the same period in the United States – where Frank Sprague contributed to the invention of an electricity collection system using overhead wires. At the end of 1887, with the aid of this system, Sprague successfully installed the first large-scale electric train system in Richmond, Virginia (Richmond Union Passenger Railway).

Horse-drawn trams are still in operation in the Isle of Man, in the Bay Horse Tramway network, which was built in 1876. Similarly, Victor Harbor Horse Drawn Tram, which was constructed in 1894, is in operation in Adelaide, Australia. New horse-drawn tram systems were created at the Hokkaido Museum in Japan, and at Disneyland.

4.9.1.3 The development of electric trams

It was not until 1914, that all the tramway networks in the world became electric. Electrification was technically perfected, and by the early 1930s the electric tram has been the main means of urban transport worldwide.

4.9.1.4 The period of dismantling of tram networks

The emergence of the private car and improvements in the level of service provided by urban buses resulted in the rapid disappearance of the tram network in most Western and Asian countries by the end of 1950. In Paris, trams were abolished in 1938. In 1949 in the United States, only 10 cities maintained tram lines.

The oldest system among all, namely the Swansea and Mumbles Railway, was bought by the South Wales Transport Company, which operated a bus fleet in the region, and it was eventually abolished in 1960.

The tram networks are no longer maintained or upgraded. As a result, the tram is discredited in the eyes of the passengers. Consequently, tram lines were slowly replaced by bus lines.

4.9.1.5 Restoration and reintegration of tramway systems

The situation began to change in favour of the tram around the mid-1980s. The 1990s marks the renaissance of trams worldwide. New modern vehicles were constructed. Their difference when compared with the old ones is such that it can be said that they constituted a brand new urban transport mode. The modern trams are longer and more comfortable,

Table 4.11 Classification of tramway systems per continent and per type (2014 data)

Continent	*Urban*	*Tram-train*	*Tourist*	*Total*
OCEANIA	4	0	6	10
AFRICA	9	0	1	10
ASIA	35	1	3	39
AMERICA	30	4	33	67
EUROPE	291	17	14	322
Total	**369**	**22**	**57**	**448**

they move almost noiselessly and are much faster, they have a modern design, and traction and braking are controlled electronically.

Nantes and Grenoble in France became the pioneer cities in the construction of modern tram systems. Their new systems were launched in 1985 and 1988, respectively.

The renaissance of the tram in North America began in 1978, when Edmonton, a city in Canada, acquired the German U2 system constructed by Sie-mens-Duewag. Three years later, the cities of Calgary, Alberta and San Diego followed.

4.9.2 Present situation

All the data recorded and analysed in what follows, relate to the year 2014. The raw data were obtained per country, per city and per line, from various available sources and cross-checked. Afterwards, they were further manipulated for the needs of this chapter. The data relate to railway systems which meet the technical and operational characteristics that are attributed to trams as described in paragraph 4.3. They serve only the city's urban space, and by majority they are referred to by one of the following terms: tram, tramway, streetcar, strassenbahn, stadtbahn. In some cases, they are referred to by the terms metro leger, light rail, but they have the characteristics of the tram as described in paragraph 4.3.

A total of 448 tramway networks in operation are recorded worldwide (Table 4.11), while 27 more are under construction.

Europe is the continent with the most tramway systems, which is approximately 72% compared to the other continents (there are 322 networks in operation in Europe alone). Most tramway networks are located in Russia (65 networks), followed by Germany (61 networks) and the United States (47 networks).

Most tourist networks are found in America (33 networks), while Europe has the most long-distance networks (17 tram-train networks).

Table 4.12 presents the tram-train systems classified per continent, country and city.

The countries which feature the largest number of tram-train networks are Germany (7) and France (5).

Out of 27 networks that are under construction, six are tram-trains.

Table 4.13 presents the urban tramway systems classified by continent and type of historical evolution as defined in paragraph 4.25.

The above data provide a rough illustration of the evolution in the construction of urban tram systems. That is, 68.3% of systems were built before 1980 and 31.7% have been built since 1980.

Urban tram systems that were built relatively recently, namely trams of categories 1 and 2, correspond to 32% (116) of the total number of urban tram systems (369).

Table 4.12 Classification of tram-train systems per continent, country and city

Continent	*Country/City*	*Number*
ASIA	Japan/Toyama	**1**
AMERICA	Canada/Calgary Canada/Edmonton USA/New Jersey USA/Seattle	**4**
EUROPE	Belgium/Oostende France/Lyon France/Mulhouse France/Nantes France/Paris France/Villejuif-Athis-Months Germany/Chemnitz Germany/Karlsruhe Germany/Kassel Germany/Mannheim Germany/Nordhausen Germany/Saarbrucken Germany/Zwickau Netherlands/Rotterdam-Hauge Portugal/Coimbra Spain/Alicante Switzerland/Bex-Villars-Bretaye	**17**
Total		**22**

In a total of 369 urban trams, 187 (51%) are in operation in just five countries. The graph presented in Figure 4.38 shows the percentage distribution of urban trams of categories 1 and 2 in relation to the floor height.

As can be clearly seen from Figure 4.38, low-floor trams have prevailed, as 79% of all recently constructed tramway networks have low-flow vehicle fleets.

Finally, 85% of modern urban tram systems feature standard track gauge. For 15% of networks that feature a different gauge, the most common values of the gauge are 1,524 and 1,000 mm.

Out of the 27 networks that are under construction, only one has a gauge other than the standard gauge (tram-train in the Spanish city of Cadiz, 1,668 mm).

Table 4.13 Urban trams per continent and per type of historical evolution

Continent	*Category 1*	*Category 2*	*Category 3*	*Total*
OCEANIA	1	1	2	4
AFRICA	3	4	2	9
ASIA	6	2	27	35
AMERICA	14	10	6	30
EUROPE	30	46	215	291
Total	**54**	**63**	**252**	**369**

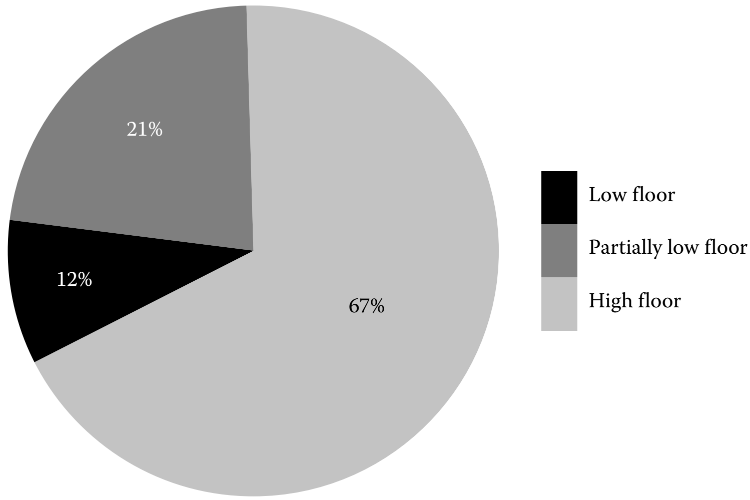

Figure 4.38 Percentage distribution of urban trams of categories 1 and 2 in relation to the floor height.

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Chapter 5

Metro

5.1 DEFINITION AND DESCRIPTION OF THE SYSTEM

The metro, or metropolitan, or sometimes termed as "underground railway" (Figure 5.1), is a system which exclusively uses electric traction and usually uses the traditional steel wheel on a rail guidance system (though sometimes rubber-tyred wheels are used, Figure 5.2), on an exclusive corridor, the largest part of which is underground and in any case is grade separated from the rest of the urban road and pedestrian traffic.

In relation to other urban transport modes, the system is characterised by

- High-frequency service (train headway up to 1 min)
- Large transport capacity (up to 45,000 passengers/h/direction) (Bieber, 1986)
- Movement, to a large percentage or the entire length, on an underground exclusive corridor
- High construction cost (€60–130 M/track-km) or even higher in some cases
- Long implementation period (in some cases even decades)

From an engineering point of view, it is a very complex and challenging project as it requires specialised knowledge regarding a variety of engineering disciplines (soil mechanics, structural mechanics, transportation engineering, architecture, power supply systems, low-voltage telecommunication systems, trackwork technologies, automated control systems, rolling stock technologies, computer systems, etc.).

5.2 CLASSIFICATION OF METRO SYSTEMS

5.2.1 Transport capacity

Based on the passenger volume they serve, metro systems are classified as follows:

- Heavy metro
- Light metro

The light metro is a hybrid solution between the heavy metro and tramway. Compared with the heavy metro, the light metro is characterised by lower transport capacity, lighter vehicles and shorter distance between intermediate stops. It is commonly selected for the service of cities with a population between 500,000 and 1,000,000 inhabitants. On the other hand, the construction of heavy metro is more appropriate for cities with a population greater than 1,000,000 inhabitants.

Figure 5.1 Athens metro system (steel wheels, driver). (Photo: A. Klonos.)

Table 5.1 compares some key constructional and functional characteristics of the two types of metros mentioned above.

5.2.2 Grade of automation of their operation

Based on the grade of automation (GoA) of their operation, metro systems are classified into four categories. Figure 5.3 illustrates these four categories and presents the operational characteristics which determine the GoA for each category (Rumsey, 2009).

More specifically

GoA1: Operation with a driver – The driver of the train is actively involved throughout the driving activity. The train is only equipped with Automatic Train Protection (ATP) system.

Figure 5.2 Lausanne metro system (rubber-tyred wheels – driverless). (Adapted from Amort, J. 2006, available online at: http://en.wikipedia.org/wiki/Rubber-tyred_metro#/media/File:Rame_m2_lausanne.JPG (accessed 7 August 2015).)

Table 5.1 Heavy metro/light metro: Basic differences as regards their constructional and functional characteristics

	Light metro	*Heavy metro*
Distance between successive stops	400–800 m	500–1,000 m
Commercial speed	25–35 km/h	30–40 km/h
Grade separation	Partial (at grade and underground)	Mainly underground
Maximum transport capacity	35,000 passengers/h/direction	45,000 passengers/h/direction
Train formation	2–4 vehicles	4–10 vehicles
Train length	60–90 m	70–150 m
Vehicle width	2.10–2.65 m	2.60–3.20 m
Driving system	With driver or automated	With driver usually or automated

GoA2: Semi-automatic Train Operation – STO. There is a supervising driver who undertakes driving only in case of system failure, and is responsible for opening and closing the doors. The train is equipped with ATP and Automatic Train Operation (ATO) systems.

GoA3: Driverless Train Operation. The train moves without a driver. There is a train attendant who is responsible for the opening and closing of the doors, and can intervene in case of system failure. The train is equipped with ATP and ATO systems.

GoA4: Unattended Train Operation. The train moves automatically and all of the above operations are performed without the presence of a driver or an attendant. The train is equipped with ATP and ATO systems.

Generally, the train operation is considered to be automatic when the trains are driverless (GoA4 and GoA3). These two GoAs must be accompanied by the installation of automatic

Grade of automation	Type of train operation	Setting train in motion	Stopping train	Door closure	Operation in event of disruption
GoA 1	ATP with driver	Driver	Driver	Driver	Driver
GoA 2	ATP and ATO with driver (STO)	Automatic	Automatic	Driver	Driver
GoA 3	Driverless (DTO)	Automatic	Automatic	Train attendant	Train attendant
GoA 4	UTO	Automatic	Automatic	Automatic	Automatic

Figure 5.3 Classification of metro systems based on the grade of automation of their operation. (Adapted from UITP. 2013b, Press kit metro automation facts, figures and trends. A global bid for automation, UITP Observatory of Automated Metros, available at: http://www.uitp.org/sites/default/files/Metro%20automation%20-%20facts%20and%20figures.pdf (accessed 14 March 2015).)

Table 5.2 Advantages (+) and disadvantages (–) of automatic metro systems in comparison to conventional metro systems (with driver)

+	Driverless → Lower operation personnel cost
+	Operation independent of the availability of drivers → Regularity and flexibility of services
+	Human factor absence → Increased traffic safety
+	Automatic driving → More costly efficient driving → Lower energy consumption → Reduced environmental impacts
+	Higher service frequency → Shorter trains for the same transport capacity → Smaller platform length
+	Unified speed, higher service frequency → Higher track capacity
+	Lower delays at the platforms, reduced time for manoeuvers at terminals → Reduced number of trains required for the accomplishment of all scheduled routes
–	Driverless → Concerning feature for some of the system's potential passengers → Discouraging factor for using the transport mode
–	Driverless → Fewer job positions
–	Increased maintenance cost and additional personnel cost for system safety associated with the automation system itself

sliding gates along the platforms (Platform Screen Doors (PSD), see Section 5.4.4) in order to increase passenger safety.

Table 5.2 shows the advantages and disadvantages of automated metro systems compared with metro systems with driver.

5.2.3 Guidance system

Based on the guidance system, metro trains are classified as follows:

- Trains with steel wheels
- Trains with rubber-tyred wheels

Figure 5.4 illustrates a bogie of a rubber-tyred metro.

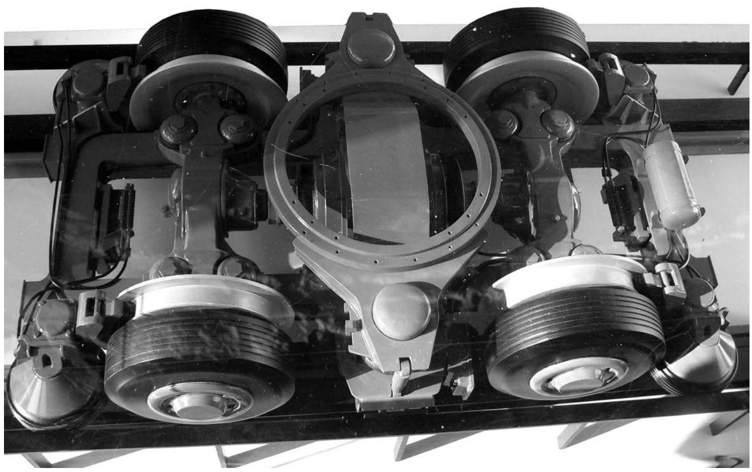

Figure 5.4 Mockup of a bogie of a M2 train. (Adapted from Rama, 2007, online image available from https://commons.wikimedia.org/wiki/File:Bogie-metro-Meteor-p1010692.jpg)

Table 5.3 Advantages (+) and disadvantages (–) of metro trains with rubber-tyred wheels and trains with steel wheels

With rubber-tyred wheels	*With steel wheels*
+ Low rolling noise	– High rolling noise
– Increased noise when starting the train	
+ Greater accelerations	– Smaller accelerations
+ Ability to move along greater longitudinal gradients (up to 13%)	– Ability to move along smaller longitudinal gradients (up 5%)
– Increased energy consumption	+ Lower power consumption
– Greater maintenance cost (frequent tyre replacement)	+ Lower maintenance cost
– Much lower lateral stability of vehicles (lateral guiding wheels required)	+ Lateral stability of vehicles
– Lower axle loads	+ Greater axle loads
+ Reduced braking distance	– Increased braking distance
+ Increased passenger dynamic comfort	– Reduced passenger dynamic comfort

Table 5.3 presents the advantages and disadvantages of trains using rubber-tyred wheels, and trains with steel wheels.

5.2.4 Other classification categories

Based on their integration in relation to the ground surface, metro systems are classified as follows:

- Underground
- At grade
- Elevated

Finally, based on the network's layout, metro systems are classified into systems that adopt

- Radial-shaped layout
- Linear-shaped layout with or without branches
- Grid-shaped layout

In most cities, the layout is mainly dictated, and thus, explained by the gradual development of the metro system, and reflects the arrangement of the city functions itself (existing or planned). For new branches (extensions) or new networks, the deployment of a grid-shaped layout is preferable and is therefore sought for implementation. The reason for this is to avoid the risk of overloading the city centre. Unlike in other cities, a radial-shaped layout is selected aiming to boost the centre. Finally, in some cities, the network is rudimentary as it only includes a single line.

5.3 CONSTRUCTIONAL AND OPERATIONAL CHARACTERISTICS OF A METRO SYSTEM

The basic characteristics of metro systems were presented in Chapter 1, Table 1.6.

In addition to those, the following are also mentioned.

5.3.1 Track layout

The alignment and, hence, the track layout characteristics of a metro line are largely determined by

- The need to serve specific locations that are trip generators, which are located at a relatively short distance from each other
- The need to deal with soil settlement when placed underground, which can be hazardous for the overlying structures

All of the above impose an alignment which largely follows the road arteries above the ground surface. This leads to the adoption of a horizontal alignment which is characterised by a considerably large percentage of curved segments and radii ranging from $R_c = 500$ m to $R_c = 150$ m. As for secondary lines (depot, sidings), radii can be reduced to $R_c = 70–80$ m.

The longitudinal profile of the line is imposed by

- The maximum longitudinal slope that a metro train can cope with
- The need to limit the depth of excavations for the stations, and the various ventilation or other equipment shafts
- The adoption of a line with a horizontal alignment profile and a longitudinal profile that allow for energy savings

The maximum longitudinal gradient of a metro network ranges between $i_{max} = 3\%$ and $i_{max} = 8\%$, though it is advisable that a gradient should not normally exceed 5%. At stations, depots and generally at locations where trains are parked, the longitudinal gradient of the track should be less than $i = 2\%$, in order to avoid possible movement of trains in case the braking system is not activated.

5.3.2 Track superstructure

The track superstructure is usually made of a concrete slab (slab track). The introduction of this track bed system instead of the ballasted track is mainly due to the following reasons:

- Much lower annual maintenance cost – easier maintenance (in case of ballasted tracks, the limited width inside the tunnels complicates the maintenance work)
- Longer life time (50 years vs. 25 years)
- Lower height of track superstructure
- Ability for road emergency vehicles to move on the track superstructure
- Better behaviour under stress – greater lateral track resistance

On the contrary, the implementation cost of slab track is greater than the cost of ballasted track (1,000–1,200 €/m as compared to 500–600 €/m, for the case of construction in a single-track tunnel).

Concerning metro systems, many techniques have been developed for slab track which differ with regard to the type and characteristics of their structural features as well as the construction and maintenance methods applied (Figures 5.5 and 5.6) (Quante, 2001; Ponnuswamy, 2004; Rhomberg, 2009). In parallel, for the same technique, differentiations are observed depending on whether the superstructure is laid in the ‘plain’ track, in areas of switches and crossings, in a depot, in twin-bore tunnels or single-bore double-track tunnels.

Figure 5.5 Slab track, Stedef system. (Adapted from Jailbird, 2005, online image available at: https://en.wikipedia.org/wiki/Railroad_tie)

Finally, special solutions are adopted for the areas where there is a need for protection against vibration and noise.

Among the first systems of slab track that were used in metro construction were the Rheda system (Figure 5.6) and the Zublin system.

The selection of a suitable system of slab track requires a multi-criteria approach. Table 5.4 gives a list of options for the main track superstructure components for the case of 'plain' track. It should be highlighted that these options are the most commonly used during the recent years, based on international construction practice.

In the last decade, a tendency to use slab track systems with direct fixing of the rails on the concrete slab is observed. This technique is gaining more and more ground in the market as due to the continuous improvement in the quality of connection between the baseplate and the rail, as well as the continuous development of materials used as elastic

Figure 5.6 Slab track: The ties of the Rheda 2000 system before they are tightened on a concrete bed, Nuremberg-Ingolstadt high-speed railway line, Germany. (Adapted from Terfloth, S. 2004, available from https://commons.wikimedia.org/wiki/File:Schwellen_Rheda.jpg 2004.)

Table 5.4 Suggested track superstructure components of slab track for a 'plain' track

Track superstructure components	*Option*	*Comment*
Rails	UIC54, C.W.R.	For $R_c < 600$ m, hardness 1,100A For $R_c > 600$ m, hardness 900A
Slab track system	Direct fastening systems without sleepers	Easy to construct and maintain
Fastenings	Spiral-shaped resilient fastenings	High elasticity and lateral resistance
Pads	Elastic rail and baseplate pads	• Plastic pads for the electrical insulation of the track • Elastomers for which the ratio of vertical dynamic to static stiffness is $K_{dyn}/K_{stat} < 1.5$ to reduce noise and vibration

pads, it has significant advantages over the classical methods of slab track using sleepers. The only drawback in the use of these systems is their moderate ability to absorb noise and vibration.

Table 5.5 proposes some options for the track superstructure components for areas that are sensitive to noise and vibrations. The system that can ensure maximum reduction in the ground-borne noise is the floating slab; however, this is also the most expensive option.

Table 5.6 attempts a comparison of the different techniques that are used to address the ground-borne noise and vibrations in urban railway systems in terms of noise reduction and ease of maintenance.

Table 5.7 attempts a comparison of the implementation cost of the above techniques. The implementation cost of a floating slab is approximately €2.5 M per km (double track).

The development of direct fixing systems which achieve a reduction of noise and vibrations similar to that achieved by the use of floating slabs (i.e. 30 dB) is in full swing. This may lead to the universal prevalence of direct rail-fixing systems over all other solutions.

A floating slab may be constructed with one of the following methods:

- With a continuous concrete slab, cast *in situ*
- With a discontinuous slab that is made of prestressed concrete elements

Table 5.5 Suggested structural slab track components for areas that are sensitive to noise and vibrations

Level of reduction of noise and vibrations	*Option*	*Comments*
Great	• Floating slab with discrete bearings (Figure 5.7) • Floating slab with elastomer strips (Figure 5.8) • Slab using springs • Floating slab with elastomer mat (Figure 5.9)	The use of floating slab with elastomer mat should be avoided due to the fact that it is very difficult to replace the elastomer in case of wear
Moderate	• Floating slab or slab with elastomer mat • Very flexible clip fastenings for the direct fixing of rails with preloading • Very flexible clip resilient fastenings for the direct fixation of rails	A techno-economic study is required for the selection of the optimal solution
Low	• Clip resilient fastenings • Rail web dampers (Figure 5.10)	A techno-economic study is required for the selection of the optimal solution

Table 5.6 Comparison of the techniques used as countermeasures for the ground-borne noise and vibrations in the case of urban railway systems

Noise countermeasure	*Noise reduction (dB)*	*Ease of maintenance*
Floating slab with elastomer mat	≈20	–
Floating slab with elastomer strips	≈25	√
Floating slab with discrete bearings	≈30	√
Floating slab with springs	≈20–25	√
Resilient fixing system	≈2–10	√√
Resilient fixing system with preloading (APT – ST)	≈10	√√
Very resilient fixing system with preloading (APT – BF)	≈20	√√
Rail web damper	≈2~5	√√

Note: Difficult maintenance, √: easy maintenance, √√: very easy maintenance.

Table 5.7 Cost factor for various noise reduction systems

Track superstructure type	*Cost factor*
Ballasted track and direct resilient fixing system of rails	1
Very resilient fixing system of rails	1.2–1.6
Floating slab	2.5–4.5

Figure 5.7 Floating slab with discrete bearings (point- like support). (Adapted from GETZNER. no date, Mass-Spring System, GETZNER company brochure, available at: http://www.getzner.com/en/downloads/brochures/ (accessed 14 March 2015).)

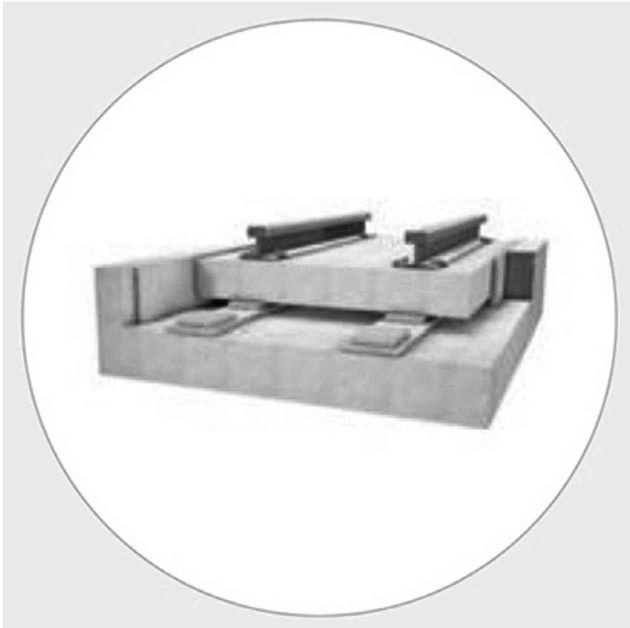

Figure 5.8 Floating slab with elastomer strips (linear support). (Adapted from GETZNER. no date, Mass-Spring System, GETZNER company brochure, available at: http://www.getzner.com/en/downloads/brochures/ (accessed 14 March 2015).)

Figure 5.9 Floating slab with elastomer mat (full surface layer). (Adapted from GETZNER, no date, Mass-Spring System, GETZNER company brochure, available at: http://www.getzner.com/en/downloads/brochures/ (accessed 14 March 2015).)

Figure 5.10 Placement of damping materials on the rail web. (From Vossloh, 2015.)

Table 5.8 provides a comparison between the two aforementioned techniques. According to the specifications of the project and the construction restrictions, the engineer should select the most advantageous solution.

5.3.3 Tunnels

Based on the number of bores (branches) and the number of tracks per bore, the underground sections of a metro network are classified into the two following categories:

- Single-bore double-track tunnels (one tunnel with two tracks)
- Twin-bore tunnels (two single-track tunnels)

The choice of the most appropriate category is affected by the geological and local conditions, that is, the available overlying area and the soil quality, as tunnels with a large diameter show greater settlement.

Table 5.8 Comparison between continuous and discontinuous floating slab

Ability to reduce ground-borne noise	The natural frequency of a discontinuous floating slab (8–16 Hz) is lower than that of a continuous floating slab (16 Hz), and therefore the discontinuous slab is more effective in reducing ground-borne noise and vibrations
Stress condition	For a discontinuous slab, additional dynamic forces are exerted on the wheel–rail contact surface as there is a change in the track stiffness due to the discontinuity
Ease of construction	The discontinuous slab can be transported with forklifts and can be fitted with the aid of load lifting systems. This implies the existence of adequate available open space as well as of appropriate access
Ease of maintenance	The replacement of elastomers for the discontinuous slab is technically much easier to achieve by lifting the prestressed slabs, provided that discrete bearings or elastomer strips have been used. In the case of elastomer mat this is not feasible, as the lifting jack can only be used locally

The construction of the metro tunnels presents certain particularities in relation to the construction of tunnels outside cities as

- Large cities are usually developed in areas with mild landscape, and soil materials at the influence depth are usually not rocky.
- The height of the soil overlying the tunnel is usually less, ranging between 10 and 25 m (as compared to even hundreds of metres of overlying soil height in the case of large tunnels in mountainous areas).
- The metro passes underneath the centre of large cities and, as a result, any visible fault (or even suspected fault) is quickly identified (with anything that this may then trigger).

The cross section of the metro tunnel excavation can be either circular or rectangular.

A metro tunnel can be constructed with one of the following methods:

- With excavation, if the project is constructed at a shallow depth beneath the road surface. The tunnelling excavation involves relocating the affected public utilities networks alongside the project as well as perpendicular to the project.
- With boring, by which open excavation is avoided. A general principle is that the upper limit of the bore that is opened must be at a distance from the ground surface that is at least the length of one diameter of the bore (e.g., about 6 m for a single-track tunnel, or 9.5 m for a double-track tunnel).
- With drilling and blasting (drill and blast). This option is very rare within city environments.

The cost of tunnelling and the comparison of costs for the different tunnelling methods depend on a large number of parameters. For shallow tunnels, construction by direct excavation is more economical compared to bored tunnelling, but for deeper tunnels, tunnelling by boring is cheaper than tunnelling by excavation.

Tunnelling with excavation is usually more economical compared to tunnelling with drilling.

In tunnels with excavation, the following two techniques are applied:

- The technique of open trench (cut and cover) (Figure 5.11)
- The technique of excavating and backfilling (cover and cut)

Regarding tunnels without open excavation, the following two techniques are mainly applied:

- The use of Tunnel Boring Machines (TBM)
- The New Austrian Tunnelling Method (NATM)

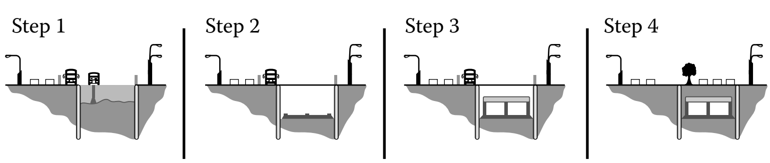

Figure 5.11 Stages of applying the cut and cover technique. (Adapted from FHWA. 2013, Technical Manual for Design and Construction of Road Tunnels – Civil Elements, Federal Highway Administration – U.S. Department of Transportation, Chapter 5 – Cut and Cover Tunnels, available at: http://www.fhwa.dot.gov/bridge/2013 (accessed 14 March 2015).)

5.3.4 Rolling stock

A typical metro train commonly consists of 4–10 vehicles.

The construction materials that are commonly used for the vehicles are semi-stainless steel and the aluminium.

The doors are divided into

- Simple sliding
- Double sliding

Regarding the interior of the vehicles, the transport capacity of the train (passengers standing and seated) is usually 600–1,200 people. The percentage of seated to standing passengers varies from 25% to 45%.

Vehicles of a metro system are equipped with conventional bogies. The bogies must be characterised by

- Small wheelbase ($2a = 1.80–2.20$ m instead of 2.50–3.00 m – used for conventional railway vehicles)
- Small wheel diameter ($2r_o = 0.70–0.80$ m instead of 0.90–1.00 m – used for conventional railway vehicles)

The equivalent conicity of the wheels and the stiffness of the springs of the primary suspension constitute the critical construction parameters of the rolling stock.

By using conventional bogies, and for horizontal alignment radii that are up to $R_c = 70–80$ m, there can be a combination of values of the equivalent conicity of the wheels and the stiffness of the springs of the primary suspension of the bogies that ensures the curve negotiation of wheelsets without wheel slip in curves on one hand, and running speeds in excess of $V_{max} > 80$ km/h at a straight path on the other hand; this is the desirable speed for straight segments. On the contrary, for smaller radii, curve negotiation of bogies is characterised by the exertion of extremely large values of the guidance forces (Joly and Pyrgidis, 1990; Pyrgidis, 2004).

5.3.5 Operation

The level of service that is provided by a metro system usually depends on the degree to which the following quality parameters are met:

- Running through areas with high demand for travel
- Short travel times
- Dense train headway/service during peak hours
- Reliability of schedule
- Appropriate pricing policy/ease in ticket supply
- Passenger safety on the train and at stations
- Air quality inside the vehicles/use of air conditioning
- Passenger dynamic comfort during transport
- Availability of passenger seats/satisfying train patronage
- Clean trains
- Accessibility/acceptable distance between successive stations
- Service for the disabled

- Integration with other transport modes/ability to provide park-and-ride services
- Passenger information regarding the route on board and at stations
- Interfaces between staff and users

5.3.5.1 Commercial speeds, service frequency and service reliability

The commercial speed of a metro system is $V_c = 30–40$ km/h, which is much greater than the commercial speed of other urban transport modes. This is because the metro moves on a traffic corridor that is made exclusively for its own use without the presence of any level crossings.

The usual practice which minimises the travel time between two successive stops is that the vehicle develops the maximum speed by accelerating as much as is possible, and then it decelerates at a slow steady pace.

In modern metro systems, the headway between trains can reach even 1 min, while the operation of trains with a headway of more than 15 min discourages the use of the metro system. Usually, a different frequency is applied during the peak hours (higher frequency) than the frequency for the remaining operational hours of the system.

The delay time for a service is usually defined as any time interval that exceeds 3 min. The halt time is the sum of two distinct time values: the time taken for the opening and closing of the doors during stopping (3 s) and the time taken for passenger boarding and alighting. Theoretically, 2 persons/s can board or alight from a door that is 1.30 m wide. For example, an approximate value of around 14 s, is required for single-track platforms for vehicles with a transport capacity of 40 persons for each door with a width of 1.30 m.

The need for dense headways imposes specific functional requirements for the signalling system, and renders the existence of an ATP system absolutely necessary.

Generally, the following systems can be distinguished:

- Conventional signalling systems
- Systems utilising telecommunications for the data transmission (Communications-Based Train Control [CBTC])

In conventional systems, the detection of trains is achieved with the aid of 'traditional' means such as track circuits (usually) or axle counters. Interlocking systems carry out route checks, control the moving components of the track (switches, flank protection), and give the appropriate signals to the driver for the route either through side signals or through cab signalling. The protection of trains is achieved either at a specific point (by acting on the brake when the driver exceeds a restrictive value) or continuously (by continuously monitoring the speed of the train). The operation of individual interlocking systems takes place at a central post (centralised traffic control) where the traffic flow is monitored by a system usually termed as Automatic Train Supervision. Often the treatment of individual systems is performed automatically by the centre's computer equipment, while inspectors are assigned with tasks, including traffic monitoring and intervention in case of emergencies (injuries, suicides, etc.).

In CBTC systems, the location of a train is monitored by the odometer of the train (regularly corrected via beacons that are placed on the track) which is usually sent to a single central interlocking system over a wireless telecommunication system. The central post issues all the commands and movement authorities for the route and the speed (thereby also integrating the protection function of the train over the same telecommunication system). The train separation principle is that of the 'moving block' which generally allows for denser headways.

The traffic control system (either conventional or CBTC-based) is integrated with various other functionalities such as passenger information systems and traffic management (computer-aided and sometimes fully automated train scheduling, with the dispatcher intervening only in cases of failures and degraded-mode operations).

5.3.5.2 Fare collection and ticket supply

An increasing number of metro systems prefer the installation of Automatic Fare Collection (AFC), in an effort to reduce ticket evasion.

An AFC system includes the following components:

- Ticket vending machines.
- Ticket offices.
- The ticket topup (add credits) machines, via which the users can purchase their ticket with the aid of an electronic card (credit card) which they can top up with additional credits (money).
- The facilities that separate the area where passengers can enter only with a validated ticket from the rest of the station (paid/unpaid areas). These facilities are usually automated gates which open when a ticket is validated ('closed system') (Figure 5.12); however, in some cases there is no physical barrier ('open' or 'honour' system) (Figure 5.13).
- Ticket sales, which are different from one system to another.
- An automated management system which consists of a central computer and is connected with all stations.

The components of the AFC system are connected electronically in order to record every single transaction.

The new fare collection systems use smart cards. These smart cards are cards that are scanned by a card reader placed near the platform entrance. The card reader records the passenger's entry point and exit point and reduces the corresponding amount for that particular trip from the card.

Smart cards were first used in Hong Kong in 1997 ('Octopus' card), and are now broadly used in many metro systems.

Figure 5.12 Automated gates which separate the area leading to the platforms from the rest of the station, West Kensington tube station. (Adapted from Mckenna, C. 2007, available online at: http://web.mit.edu/2.744/www/Results/studentSubmissions/humanUseAnalysis/jasminef/ (accessed 7 August 2015).)

Figure 5.13 Ticket validation systems without physical barriers, Akropoli Metro station, Athens, Greece. (Adapted from http://www.athenstransport.com/english/tickets/ (accessed 14 March 2015).)

Latest developments in AFC technology include direct ticket payments through bank credit cards or mobile phones.

5.3.5.3 Revenues for the system operator

Granting rights to third parties for managing areas for commercial use within metro stations may constitute an additional profitable source of income for the operating company, added to the fare revenues. There should, however, be set limits regarding the operation of these sites. Proper initial planning and proper management of the system are required, so as to not interfere with the primary objective of the stations, which is the unhindered and safe movement of passengers.

The selling of advertising rights at stations and metro trains can also bring significant income to the system operating company. There are two ways of advertising in a metro system:

1. Fixed advertising (advertising signs – Billboards placed on the platforms and at locations from which passengers frequently pass when moving within the station; display of advertising messages on tickets; billboards inside and outside of trains).
2. Variable advertising – Variable (mainly visual) advertising is done with the aid of modern electronic devices. Screens and high-definition televisions in various forms (LED, PDP) are used; these can be placed at the platforms, at strategic locations in the stations, and inside the trains.

5.3.6 Implementation cost

The implementation cost of a metro is very high (€60 M–€130 M per track-km, 2014 prices, and in some cases much higher). The variation of the cost is high, as it is affected by a number of parameters such as (Davies, 2012; MacKechnie, no date).

- The percentage of network length that is underground, above ground (elevated) or at grade
- The excavation method used (deep bore or cut and cover)

- The type of the cross section of the tunnel (twin-bore tunnel or single-bore double-track tunnel)
- The quality of the subsoil
- The depth of the line
- The number of stations
- The length of the platforms
- The expropriations and land values
- The labour cost and the material costs in each country
- The technologies used for the various electromechanical and railway systems, and the rolling stock

The average cost of building a metro system, with a percentage of underground length of 75%, varies between €70 M and €90 M per km. For a 100% underground metro system, the cost varies between €100 M and €130 M per km.

In recent years, there has been a trend of constructing metro systems with the smallest possible percentage of underground length in order for the cost of construction to be as economical as possible.

The cost of the rolling stock varies between €1.3 M and €2 M per car (2014 prices), so, for example, for a six-car train the cost is approximately €8–12 M, depending on the train length, width, passenger capacity, internal fittings, kinematic characteristics and performances, driverless/with driver and so on.

The driverless metro systems require higher investments for the supply of the rolling stock, the control systems and the protection systems both at the platforms and on the track (UITP, 2013a; *Metro Report International Journal*, 2014). On the contrary, the construction cost of the stations may become less, as the possibility of more frequent service allows reducing the length of platforms. However, in automated systems, the operational costs are almost half compared to the conventional ones (cost reduction for personnel, energy consumption reduction, but increased maintenance cost for protection systems).

5.4 METRO STATIONS

Metro stations are divided into three categories according to the functions they serve:

- Simple stations, whose only mission is to serve the area surrounding the station
- Transfer stations, serving transfers between lines of the same metro network
- Interchanges, where there is connection with other transport modes (trams, buses, suburban rail services, etc.)

The stations constitute structural components of the system that are not usually constructed with the TBM. This is due to their usually rectangular cross section, their different dimensions with respect to the tunnels and, most importantly, due to their own different cross section transversally to the alignment. The stations are usually constructed by excavation which requires the occupation of space on the ground surface, with all that this entails for the traffic in the city and for the activities of its inhabitants.

In this context, it is imperative that a complete study of the system's stations be performed before the beginning of the construction of the metro system. This is because, if certain construction and design options are not carefully and appropriately considered, there is an increased risk of failures and malfunctions either directly or in the mid-term; this will result in actual construction cost that is significantly higher than the initially estimated cost.

It should be noted that the construction of stations increases the total construction cost by 25%–30%, while the construction of an underground station is 4–6 times more expensive than the construction of a surface station.

Three of the main design/construction issues of a metro system that are of concern to the engineers during the phase of the project construction are

- The location/selection for stations
- The depth of their construction
- The method by which a station is constructed, as well as some of its structural elements

The above three construction criteria are influenced by many parameters which influence one another, a fact that renders the selection a difficult task. The adoption of a suitable solution is a matter of knowledge, study and research; however, the experience gained from similar projects remains an irreplaceable asset for both designers and constructors.

5.4.1 Location selection for metro stations

The location of metro stations is studied in accordance with the servicing of network users and, generally, the servicing of areas where there is a high travel demand. The unsuitable selection of the locations of stations can lead to failures and malfunctions such as increased walking time for pedestrians, lack of service for locations that constitute transport generators, unsuitable service for areas with increased travel demand (universities, stadiums, hospitals, etc.) and inability to service 'park-and-ride' facilities. Finally, some external factors arising from the location of the stations (such as the expropriation of areas where the stations will be built), if not properly addressed, they may result in delays in the stations' construction as well as in an increase in the construction cost.

The location selection for the metro stations depends on

The trip characteristics of potential users: One of the issues that need be addressed initially is to determine the number of persons who want to travel, where they want to go, when and how often. To collect this information, appropriate transport studies are necessary. The selected location of the stations must serve the travel demand.

The accessibility of stations: The stations should be placed at intersections of major roads, close to squares, at locations of mass entertainment (stadiums, shopping centres), hospitals, universities and public services.

The availability of space for the construction of metro stations: The stations should be located in areas of the city where there is available surface area for the installation of the construction site and the performance of the excavation, even temporarily.

The distance between stops: Maintaining an average and acceptable distance between two successive metro stations is necessary. This distance should be shorter at areas where population density is high, and it should be longer at areas where density is low.

The land uses: Identification studies of land uses along the alignment of the metro system are necessary. The stations should be located in areas where land uses justify their presence and require the presence of a high-capacity transport mode such as the metro.

The design of the alignment of the line: The optimum solution should be obtained with regard to the geometry of the alignment (both in horizontal alignment and in vertical profile), while at the same time it should be designed so as to achieve the maximum commercial train speed.

The connection to other public transport modes: Metro stations should be placed in locations that allow the transfer to other transport modes and, generally, locations that ensure complementarity among the available transport modes.

The terminals: Finally, with regard to terminals, these should be installed in places that enable easy connection of the metro with other modes of transport, such as railway stations, airports and bus stations, while ensuring that there is enough space for 'park-and-ride' services which are essential for a high level of service. The track layout of the metro terminals must allow the performance of the necessary train manoeuvering and the connection with areas of depots where trains will be parked, maintained, repaired and cleaned.

5.4.2 Construction depth of metro stations

The construction depth of metro stations should be selected carefully taking into account all related parameters such as the tunnel's depth for safe tunnelling, the ground conditions, the area-specific characteristics (land availability, public utility networks, archaeology, neighbouring and overlaying buildings, etc.), while an unsuitable construction depth can have adverse consequences such as minor or major impacts or even damages to neighbouring and overlying buildings, structures or networks, increased effects from the presence and pressure of the ground water (resulting in continuous pumping of water during construction, or to the overdimensioning of the system's components), the possible crossing with public utilities networks and archaeological findings (resulting in a large increase in cost of implementation and duration of construction). The worst possible impact that may occur is the uncontrolled subsidence of the overlying soil and ground during the excavation or during the construction of the stations, leading to possible damage to any overlying buildings.

More specifically, the construction depth of metro stations depends on the following:

The characteristics of the soil: The necessary geotechnical/geological studies and tests (both *in situ* and laboratory) must be performed in order to determine the characteristics of the subsoil and the presence of groundwater zones in the area where it is planned to construct the station. These studies largely determine the technical feasibility of the project in relation to the construction depth.

The archaeological findings: The presence and relative risk of archaeological zones should be estimated by competent archaeological services so as to avoid excavations at those areas, or perform them at a greater depth, if possible.

The public utility networks: Public utility networks, which include water supply, sewage sanitation, gas supply and liquid fuels, electricity grid and telecommunications network, are present in all major cities. The design and construction of a metro system must respect their presence, the complexity of their structure, the identity of their construction and their behaviour during the construction and operation of the project.

The overlying buildings: The passage of a metro line under sensitive, old or even listed buildings has significant drawbacks. If such a design cannot be avoided, it is required to perform studies to assess the anticipated displacements and subsidence of the ground surface and the impact of these displacements on the static behaviour of all the structures within the zone of influence.

Seismicity: In earthquake-prone areas, the construction of a metro network is of a special nature. It is needed to thoroughly examine the influence of the presence of underground structures (tunnels, stations, etc.) on the surface ground acceleration. The maximum and minimum acceptable values of certain parameters, such as the depth of the tunnels and the horizontal distance between an overlying structure and the tunnel axis, beyond which the difference in the surface ground acceleration cannot be neglected, should also be taken into account in all the structural risk analysis undertaken for the buildings and structures within the metro works zone of influence.

5.4.3 Construction methods

The construction methods that are applied for metro stations concern the following structural elements of a station:

a. The construction of the station's 'shell'
b. The surface constructions and particularly
   - The station's entrances
   - The protrudings from the ground of elements used for ventilation or other electromechanical structures
   - The lifts to the street level
c. The number of levels of the station
d. The stations' architecture, related to the functional and operational design of stations

5.4.3.1 Construction of the station's shell

Initially, the working site is installed, the preparatory work is performed in the site area, and the public utility networks are relocated in order to release the necessary space for construction activities. In continuation, archaeological excavations are carried out down to the level where no more archaeological findings exist. Then, the construction of the temporary civil works can proceed depending on the construction methodology (piles, sheet piles, diaphragm walls, etc.), although in some cases if the archaeological excavation is deep, for example, 5–10 m, these temporary civil work structures are necessary in order to enable the archaeological investigation itself to be carried out.

If the construction methodology is of the 'cut and cover' principle, and once all the station excavations are completed down to the lowest level, then waterproofing and concreting of the permanent civil works structures begin with a 'bottom-up' sequence, and this is carried out up to the roof slab of the station, after which the ground surface is finally reinstated.

If instead, a 'top-down' construction methodology is followed, the sequence starts by constructing the peripheral piles or diaphragm walls, followed by the archaeological excavations. The construction activities are then followed by the roof slab construction by *in situ* concreting, then the excavations down to the first underground level are performed, followed by the first-level slab construction, then the excavations down to the second level are performed, followed by the second-level slab construction, and this sequence is followed until the concreting down to the lowest level. With this methodology, when reaching the lowest level, the civil works construction is fully completed as well.

Alternatively, in an NATM-type of station construction method, a shaft is constructed, occupying a relatively small surface area at street level, and access is made possible through that vertical shaft in order to construct the station platforms in a fully underground manner. As a consequence, NATM-type stations are usually deep stations. The original shaft, constructed by cut and cover from the street level, is typically used to house the concourse and electromechanical equipment areas of the station.

5.4.3.2 Surface construction

Entrances: The design and location selection of all surface structures must be coordinated with the existing features of the surface conditions and with any future interventions that are planned by any public or local authorities. In fact, the study of surface structures must be coordinated with existing and future structures relating to

- Provisions of roads, sidewalks and pavements
- Transfer facilities to and from other modes of transport, such as bus stops, taxi ranks, car parks and so on
- Buildings
- Public utility networks
- Arrangements of public spaces, parks and gardens, reinstatement of the areas near the station

The requirements for the location of the stations' entrances are

The provision of direct access: The exact location of entrances should ensure an easy layout for vertical communication and the elimination of underpasses, or at least the minimisation of their length to the greatest possible extent.

The penetration of natural light: The greatest possible direct vertical communication between the street level and the public reception area, allows the penetration of natural daylight (Figure 5.14), which is desirable, as it contributes to improving the quality of space at the public reception level, and ensures the smoothest possible transition from the natural lighting of the exterior spaces, to the dark underground areas of the metro.

The integration with the existing conditions of the surface: The entrances shall be located so as to protect and enhance the natural and built environments, while at the same time they must not disturb the road traffic and the movement of pedestrians. The siting of station entrances is usually performed

- In existing open spaces (public squares or small parks)
- On sidewalks, if the required space is available
- In existing buildings (Figure 5.15)

Figure 5.14 Metro station entrance made of Plexiglas, Canary Wharf tube station, London, UK. (Adapted from Chmee2, 2013, available online at: https://commons.wikimedia.org/wiki/File:Canary_Wharf_tube_station_in_London,_spring_2013_(3).JPG (accessed 7 August 2015).)

Figure 5.15 Entrance to the Angel station, London Underground, UK. (Adapted from Sunil060902, 2008, available online at: http://en.wikipedia.org/wiki/Angel_tube_station (accessed 7 August 2015).)

The entrances can be designed appropriately in order to be integrated with existing buildings. In such cases, the cost of land acquisition, and the difficulties and construction cost, as opposed to the expected benefits, should be taken into account.

The integration in the city's historic sites: Particular attention should be paid to the location and the design of entrances in areas of archaeological interest and in areas where archaeological findings may be revealed during the construction of the metro. In such cases, in addition to the necessary coordination with the competent authorities, the study of the entrances must also be coordinated and integrated harmoniously in the layout of the archaeological sites. Also, a special study is required for the location and configuration of the stations' entrances in places that are of particular importance within the historical centre of the city. In such cases the entrances should be designed so as to blend with the exterior appearance of traditional buildings.

5.4.3.3 Number of station levels

The number of levels of metro stations depends on the needs and specific functional and operational characteristics of each station separately. Stations may have from one to several (e.g., five) underground levels depending on the station design. Starting from the top, these levels usually contain

- Street level, where various covered or noncovered accesses to the station are located, together with emergency exits and ventilation openings/grilles.
- Public reception area level (usually called 'concourse level') – with passenger movements and ticket purchasing/validation. Staff rooms are usually located on this level too.
- Electromechanical and railway systems – technical areas equipment level.
- Platform level.
- Under platform levels including cable network and piping network corridors, pumping rooms/sumps, etc.

Between the concourse level and the platforms there may be a 'transfer level' for transfer to another metro line, if the station serves more than one line, or simply as an intermediate level in deep stations. Also, stations can be designed so that electromechanical equipment areas may be isolated at one or more separate levels to avoid housing together with the passenger movement areas, or could be blended within the station levels together with the public areas.

The construction of an underground project, such as the metro, requires not only the construction of conventional stairs but also the installation of escalators and lifts for facilitating the vertical movements of passengers.

According to statistics, the level of service to be provided by the escalators is typically

- 60 passengers/min/m of width of the escalator moving to an upper level.
- 75 passengers/min/m of width of the escalator moving to a lower level.
- The capacity limit of escalators is 135 passengers/min/m of width.

Besides escalators, moving walkways (travelators) are also used and are particularly useful for the faster and more comfortable movement of the public within the same level of a metro station. The desired level of passenger service is

- 100 passengers/min/m width of moving walkway

Finally, lifts are used in stations with one or more levels, with one or more lifts leading from the street level to the ‘unpaid’ part of the concourse area, and more lifts leading from the concourse area to the platforms (typically two for the case of side platform stations, that is, with one lift per platform and one lift only in the case of centre platform stations.

5.4.3.4 Station architecture

Each station should function and operate providing comfortable and safe stay and movement to its users, bearing in mind the requirements for adaptability to the particular environment of integration for each case. The promotion of a specific and special identity for the station in conjunction with the standardisation, and the originality of styles and colours contributes decisively to the attractiveness of the system. Regarding the standardisation of the architecture of metro stations three alternatives are adopted

- Standard principles of construction and architectural expression, which can then be adapted to each different type of station. In this case, the key structural and architectural features of each station are uniform. Appropriate adjustments to the needs of each station provide each station with a relative specificity.
- Separate architectural design for each station based on its functional, structural and morphological specifications. In this case, the system as a whole, acquires the necessary uniformity through the use of the same materials and the use of standard signposting.
- Design of a typical station. This solution is used in case there are no requirements for a special architecture. However, the ‘typical’ station only refers to the architectural approach as the different station depths, geotechnical conditions, location of entrances at street level, functional requirements of electromechanical systems and so on never lead to ‘typical’ station concepts.

In all cases, the use of finishing materials that are easy to maintain and replace is required. Figures 5.16 and 5.17 illustrate the architecture that was chosen for metro stations of European metropolis.

Figure 5.16 Port Dauphine station entrance, Paris, France. (Adapted from Clericuzio, P. 2004, available online at: http://en.wikipedia.org/wiki/Paris_M%C3%A9tro) (accessed 7 August 2015).)

Figure 5.17 Solna Centrum station, Stockholm, Sweden. (Adapted from Halun, J. 2013, available online at: https://commons.wikimedia.org/wiki/File:20130601_Stockholm_Solna_centrum_Metro_station_6879.jpg (accessed 7 August 2015).)

5.4.4 Platforms

Platforms are the areas where boarding and alighting of passengers to and from the trains take place. At the same time, platforms also serve as waiting areas for the passengers.

5.4.4.1 Layout of platforms

The platforms can be placed either between the two main tracks (central platform), or at both sides of each track (side platforms) Figures 5.18 and 5.19, while side platforms can also be placed on entirely different levels within a station.

The layout of the central platform is the most economical solution; however, it often causes jams in passenger flows while requiring very careful marking to guide and orientate passengers.

In some stations, the central platform is accompanied by two side platforms. Although this solution requires more space, it is functionally ideal because it allows the boarding of passengers from the central platform and the alighting of passengers on the side ones.

Figure 5.18 Metro station – Central platform, Athens, Greece. (Photo: A. Klonos.)

5.4.4.2 Platform dimensions

The width of the platform is determined by the anticipated traffic during peak hours. The minimum width of the platform is 2.50 m while the usual width is between 3.50 and 4.00 m. Greater widths are foreseen for busy stations.

In the area of the platforms, no columns should be present as they obstruct passenger movements (Figures 5.20 and 5.21) and reduce visibility.

Depending on the platform's use, its surface can be divided into the following zones:

- The safety zone, with a width of 0.50 m measured from the edge of the platform which should not be used.
- The concentration zone, which is used by passengers waiting to board the trains. The density of passengers in this zone is estimated at 2 persons/m^2.
- The traffic zone, located behind the concentration zone, with a width of about 1.50 m for the movement of passengers alighting from the trains.
- The equipment area, which is actually the remaining width of the platform. Cash desks, electronic ticketing distributors and so on are placed in this zone.

Figure 5.19 Metro station– Side platforms. (Photo: A. Klonos.)

Figure 5.20 Central metro platform with columns, Athens, Greece. (Photo: A. Klonos.)

The total width of the platform depends to a large extent on the importance that is given to the concentration zone in relation to the variation in the number of passengers boarding the trains.

Regarding the height of the platform, it should be such that when the train is stopped, the vehicle's floor is on no occasion lower than the level of the platform's floor.

The gap between the floor of a stopped vehicle and the platform, must be minimised. Usually this gap should be a few centimetres wide and no more than 5 cm.

Finally, as mentioned in Section 5.2.2, for driverless metro systems, automated sliding gates are usually installed along the platform called platform screen doors (PSDs) or platform edge doors. Based on this technology, the platform is separated from the tracks by these transparent doors. The doors of the train and the platform open simultaneously, and only when the train is stopped at a prespecified position. These partition doors, depending on their height from the platform floor may be 'half height' (Figure 5.22; PSDs height

Figure 5.21 Side platform with columns, Sofia, Bulgaria. (Adapted from Krussev, E. 2012, *A Walk through Sofia's New Metro Tunnel* (*Part 1*), Online image, available at: http://www.publics.bg/en/publications/112/A_Walk_through_Sofia%E2%80%99s_New_Metro_Tunnel_(Part_1).html (accessed 25 January 2012).)

Figure 5.22 Automated gates at metro stations (half height), Ōokayama station, Tokyo, Japan. (Adapted from Tennen-Gas, 2008, available online at: https://commons.wikimedia.org/wiki/File:Platform_screen_doors_003.JPG (accessed 7 August 2015).)

Figure 5.23 Automated gates at metro stations (full height), Copenhagen, Denmark. (Photo: A. Klonos.)

approximately 1.5 m) or 'full height' (Figure 5.23; PSDs height approximately 2.2 m). Depending on the platform design, the space above the PSDs may be left open (Figure 5.23) or may be closed with suitable architectural panels up to the platform ceiling, thus completely isolating the platform and track areas (Figure 5.24).

5.5 DEPOT FACILITIES

Metro depots, depending on the activities performed, are distinguished as fully operational and partly operational.

In a metro network, as in the case of trams, the depot is of particular importance for the efficiency of the system.

The metro system is characterised by continuous extensions and dynamic adaptation to the passengers' transport requirements, resulting in changes in the fleet size and, consequently, changes in its maintenance requirements. Therefore, while during the construction of a new metro system, the construction of a fully operational depot constitutes common practice; this is not the case with the depots that are constructed at a later stage to serve the network expansions.

Figure 5.24 The platform screen door of Ecological District Station of Kaohsiung MRT (full height – isolated platform from tracks). (Adapted from Shack, 2008, online image available from https://commons.wikimedia.org/wiki/File:Platform_screen_door_of_Ecological_District_Station.jpg)

When there is more than one depot in a network, an allocation of the performed activities among the available depots is very often the selected option by the system operator.

Depending on their number and their location in relation to the main line of the network, some depots perform only specific activities (e.g., only parking, light maintenance or inspection), while others offer full-scale maintenance facilities.

The first stage of the design of a new metro depot is the selection of the area where it will be built. The geographic position of the depot is selected based on the following criteria:

- Sufficient ground plan area
- Acceptable length of dead vehicle-kilometres
- Slight landscape (small height variations across its area)
- Availability of suitable land
- Ability to integrate the depot into the existing land use – ability to access the road

The planning and dimensioning process is difficult due to the great disparity between different metro systems and the lack of standards.

Metro depots exhibit major differences in comparison with tram depots. More specifically

- Different horizontal alignment radii R_c are adopted. For tramway depots, a minimum horizontal alignment radius of $R_c = 17–18$ m is used, preferably $R_c = 20$ m, while for metro depots, a radius of $R_c = 70–80$ m is used.
- The length of parking and maintenance tracks in the respective areas is different between the two depot types, since the metro trains are longer (60–150 m) than those of the tramway (30–45 m). However, it should be clarified that metro vehicles can be detached from the train and can be led individually to the maintenance tracks.
- In the case of the metro, the depot is usually located close, yet outside the urban area, as opposed to the case of the tramway, where it is normally required to search for an area within or at the boundaries of the urban area.
- The metro network usually has a radial shape. This allows for more options in searching for an area for the construction of the depot. The tramway network usually has a 'linear' shape which reduces the options for the depot site considerably.

Figure 5.25 Metro depot – Entry and exit in parking tracks from the same side. Bidirectional train movement by necessity.

- The two depots require different facilities for the maintenance of the bogies (due to the different floor heights of the vehicles). The larger size of the engineering equipment that is used at the metro depot and the higher number of electronic and other systems, in comparison with the tramway, result in a requirement for larger maintenance areas and more staff.
- The comparatively large horizontal alignment radii that are adopted at metro depots render the construction of the ring track (loop line), which interconnects the various facilities, more difficult, compared to tramway depots. This fact mainly imposes the entry and exit to the parking and maintenance area from the same area (i.e. through bidirectional train movement, Figures 5.25 and 5.26). Also, having an access redundancy, that is, with two entry/exit points for the trains, is considered as a significant operational advantage.

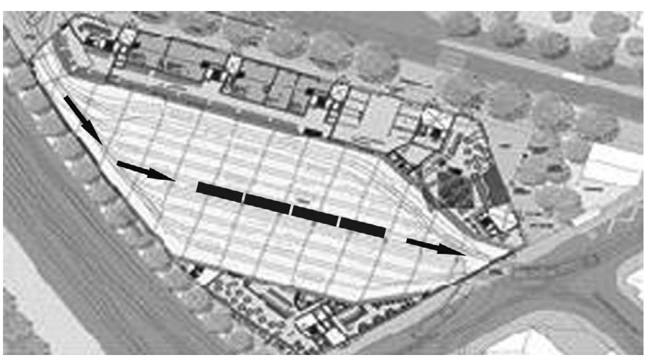

Figure 5.26 Tramway depot; entry and exit in the parking tracks from the opposite side.

Table 5.9 Estimated ground plan area per traincar for the various facilities and for the depot in total

Facility	*Estimated required ground plan area per traincar (m^2)*
Parking area	432.34
Maintenance area	196.55
Warehouse	84.23
Management – staff buildings	38.22
Private car parking area	126.57
Total ground plan area	3,546.63

Source: Adapted from Pyrgidis, C., Chatziparaskeva, M. and Siokis, P. 2015, Design and operation of tramway and metro depots – Similarities and differences, Approved for presentation at the International Conference Railway Engineering 2015, 30/06–1/07, 2015, Edinburg.

As in the case of tramway systems, the estimate of the depot's ground plan area is an important tool in the selection process.

Pyrgidis et al. (2015) attempt an estimate of the required ground plan area of the premises and facilities of a metro depot with the aid of data collected from metro depots that are either existing or under construction. More specifically, after statistical analyses, the average values of the surface area of the individual installations and of the total ground plan area of the depot per train of the design fleet were calculated.

Table 5.9 provides the results obtained from this process.

5.6 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

The metro system is a mass transit system which features many advantages that can be instrumental in improving a city's level of transport service.

However, because of the particularities of its construction (underground work) and the high implementation cost, in order to determine the feasibility of the construction, an extensive feasibility study must be carried out in advance.

More specifically, the feasibility study of a metro network includes

- The examination of the urban characteristics of the study area
- The analysis of trip characteristics, that is, the assessment of the travel demand, the determination of the type of movements, the origin and destination points and so on
- The examination of the existing public transport systems, that is, the examination of the level of service that they provide and the possible solutions toward the improvement of their operation
- The analysis and assessment of each transport system solution's impact on the city
- The choice of alternative alignments and their evaluation according to technical, financial, operational and environmental criteria
- The assessment of the socio-economic efficiency of the project

The metro is usually selected as a transport mode for cities with populations greater than one million inhabitants and in cases where

- There is a high demand for travel (>10,000 people/h/direction).
- There is a persistent specific air pollution problem in a city or region.

- The other public transport systems (e.g., buses) operate in saturation conditions and despite any improvements (routing, use of longer vehicles) they fail to meet the current demand in offering a good level of service.
- There is available funding.
- There are spaces available in the system's periphery for the installation of terminals of the system itself and the creation of parking areas and bus terminal facilities.

5.7 HISTORICAL OVERVIEW AND PRESENT SITUATION

5.7.1 Historical overview

The first metro system in the world was built in London. More specifically, in 1863, the first tunnel was excavated in the city centre, for the railway connection between Paddington and Farringdon. The first 'dedicated' metro line, however, was a line that was constructed at the City (South London), between Stockwell and King William Street. This line was launched on November 4, 1890, and is now part of the Northern line of the London Underground. This line was the first electrified underground line in the world.

Metro systems exist and operate worldwide, while numerous new metro systems and expansions of existing ones are under construction.

5.7.2 Present situation

A total of 155 cities with operating metro systems are recorded in a total of 56 countries, while 39 systems are under construction.

These data and the data recorded and analysed in the following sections relate to the year 2014. The raw data were obtained per country, per city and per line, from various available sources, and cross-checked. Afterwards, they were further manipulated for the needs of this chapter.

It should be noted that in the case of cities where there are metro systems that are managed by different operators, these systems were registered as one system. The data refer to urban rail systems that commenced operation before 1st of January 2015 and meet the technical and functional characteristics attributed to metro systems as described in Section 5.3. These systems

- Mainly serve the urban centre of a city
- Move underground for the largest part of their route
- Develop a maximum running speed of $V_{max} \leq 110$ km/h, while their commercial speed varies between $V_c = 25$ and 50 km/h

The majority of the above systems are referred to in the city in which they operate, by the terms 'Metro, Subway, Underground, Metropolitan Railway, Metrorail, U-Bahn'. In some cases they are referred to by various terms or brands such as 'Rapid Transit', 'Rail Transit', 'Sky Train', 'Light Rail Transit System', 'Tren Urbano', but they all feature the characteristics of the metro system, described above.

Table 5.10 provides the percentage distribution of cities with metro systems per continent. The graph presented in Figure 5.27 illustrates the percentage distribution of cities with metro systems per country.

It should be noted that out of a total of 56 countries, the 12 top ones have 64.62% of the total number of metro systems. The prevailing track gauge is 1,435 mm.

Of the 39 new under construction metro systems, 31 are in Asia, of which 12 in China. Table 5.11 presents the 10 cities with the largest metro network length worldwide. It should

Table 5.10 Percentage distribution of cities with metro systems per continent

Continent	*Number of cities with metro systems*	*Percentage (%)*
Africa	2	1.29
America	33	21.29
Asia	61	39.35
Europe	59	38.06
Total	155	100

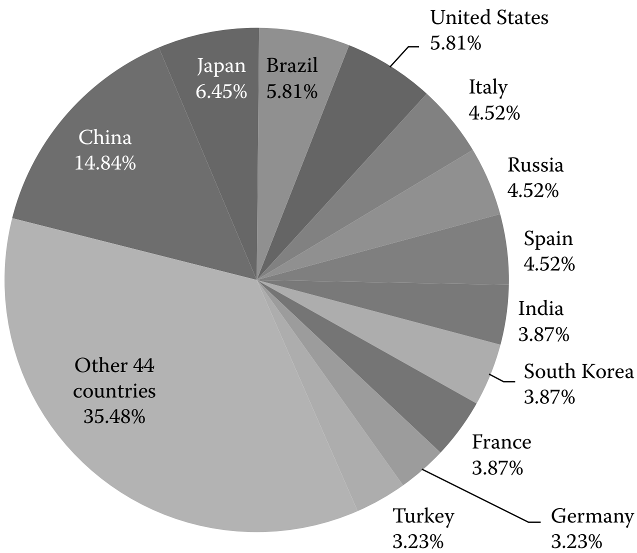

Figure 5.27 Percentage distribution of cities with metro systems per country.

Table 5.11 Ten cities with the largest metro network length in the world

No.	*Country*	*City*	*Total length (km)*
1	China	Shanghai	538
2	South Korea	Seoul	468.9
3	China	Beijing	465
4	United Kingdom	London	402
5	United States	New York	373
6	Russia	Moscow	325.4
7	Japan	Tokyo	316.6
8	Spain	Madrid	293
9	China	Guangzhou	240
10	Mexico	Mexico City	226.5

Table 5.12 Metro systems with a GoA4 grade of automation

No.	*Country*	*City*
1	Brazil	Sao Paulo Metro
2	Canada	SkyTrain (Vancouver)
3	Hong Kong	Hong Kong MTR
4	China	Copenhagen Metro
5	France	Rennes Metro
6	France	Toulouse Metro
7	France	Lille Metro
8	France	Lyon Metro
9	France	Paris Metro
10	Germany	Nuremberg U-Bahn
11	Italy	Brescia Metro
12	Italy	Milan Metro
13	Italy	Turin Metro
14	Singapore	Mass Rapid Transit
15	South Korea	Seoul Metropolitan Subway
16	South Korea	Busan Metro
17	Spain	Barcelona Metro
18	Switzerland	Lausanne Metro
19	Taiwan	Taipei Metro
20	UAE	Dubai Metro
21	Malaysia	Kuala Lumbur
22	Japan	Tokyo
23	South Korea	Yongin
24	Japan	Kobe
25	USA	New York
26	South Korea	Uijeongbu
27	Japan	Yokohama
28	Japan	Nagoya (Aichi)
29	Japan	Osaka
30	USA	Las Vegas
31	USA	Miami
32	USA	Detroit

Source: Adapted from Wikipedia. no date, a, available at: http://en.wikipedia.org/wiki/List_of_driverless_trains (accessed 14 March 2015).

be noted that the city with the smallest metro network length is the city of Catania in Italy; the length of its metro network is only 3.8 km.

Table 5.12 presents the metro systems that operate either exclusively or partially at a GoA4 (Wikipedia, no date, a).

Table 5.13 presents the metro systems that operate with vehicles that feature rubber-tyred wheels for at least one of their lines (Wikipedia, no date, b).

The graph presented in Figure 5.28 illustrates the evolution of the construction of metro systems from year 1880 until today. More specifically, from 1890 until 1970 (8 decades), a total of 32 metro systems were constructed, while from 1970 until today, a total of 123 systems were constructed.

Table 5.13 Metro systems using vehicles with rubber-tyred wheels

No.	*Country*	*System name*	*Line(s) featuring trains with rubber-tyred wheels*
1	Canada	Montreal Metro	All (4/4)
2	Chile	Santiago Metro	Line number: 1,2,5 (3/5)
3	France	Lille Metro	All (2/2)
4	France	Lyon Metro	Line code: A,B,D (3/4)
5	France	Marseille Metro	All (2/2)
6	France	Paris Metro	Line number: 1,4,6,11,14 (5/16)
7	France	Rennes Metro	All (1/1)
8	France	Toulouse Metro	All (2/2)
9	Italy	Metrotorino	All (1/1)
10	Japan	Hiroshima Rapid Transit (Astram Line)	All (1/1)
11	Japan	Saporo Municipal Subway	All (3/3)
12	South Korea	Busan Subway	Line number: 4 (1/5)
13	Mexico	Mexico City Metro	(10/12)
14	Switzerland	Lausanne Metro	Line code: M2 (1/2)
15	Taiwan	Taipei Metro	Brown Line (1/11)

Source: Adapted from Wikipedia. no date, b, available at: http://en.wikipedia.org/wiki/Rubber-tyred_metro (accessed 14 March 2015).

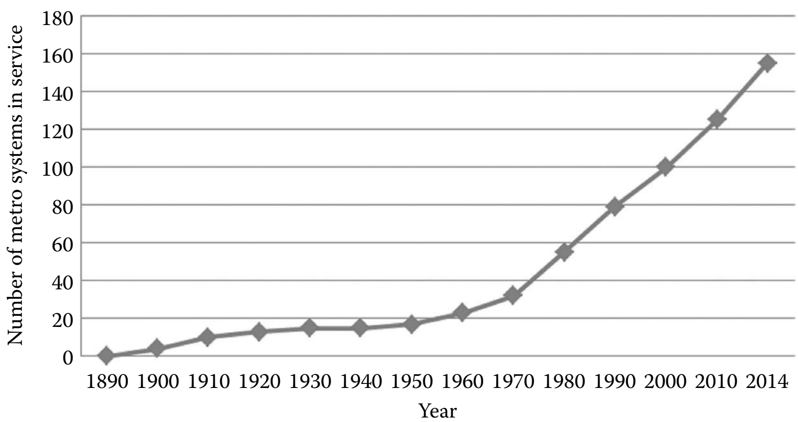

Figure 5.28 Time evolution of commencement of operation of metro systems.

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Chapter 6

Monorail

6.1 DEFINITION AND DESCRIPTION OF THE SYSTEM

The monorail is an electrified light rail passenger transport system. This transport mode (in a typical manner, an articulated train) is formed with a small number of vehicles (2–6), and in most cases it moves via rubber-tyred wheels, on an elevated permanent way (guideway). The guideway is essentially a beam, which takes over the traffic loads and guides and supports the vehicles (guidebeam).

The system often covers short distances (in the range of S = 10 km). It develops maximum running speeds of V_{max} = 60–90 km/h and commercial speeds of V_c = 20–40 km/h. It is especially offered for transport within leisure places (thematic parks, zoos, etc.) due to the panoramic view it allows its users, but they have also been recently introduced as air–rail links, and as a means to circumvent land scarcity issues in congested cities (e.g., in China, in Indonesia, and in South Korea).

6.2 CLASSIFICATION OF THE MONORAILS AND TECHNIQUES OF THE SYSTEM

6.2.1 Train placement on the guidebeam

Depending on the way the trains are placed on the guidebeam, monorail systems fall into three categories:

- *Straddled systems*: In straddled systems, the train sits above a beam, surrounding it (Figures 6.1 and 6.2). The guidebeam's profile is of either orthogonal form or type I. The vehicle 'embraces' the beam, thereby providing safety against derailment.

  The train moves with the aid of direct current (DC) electric motors that are placed between the vehicles. Those motors trigger a system of either perpendicular or transverse wheels in rolling relatively to the guidebeam.

  The wheels are connected with the car body through an electronically controlled active suspension. This levels the vehicle's floor to the station's platform, thus allowing the safe boarding and alighting of passengers.
- *Suspended systems*: In the case of suspended monorails (Figure 6.3), the vehicle is placed (suspended) below the guidebeam. The suspended systems produce greater potential visual intrusion, since the total height of the vehicle and the guidebeam is approximately 6 m.

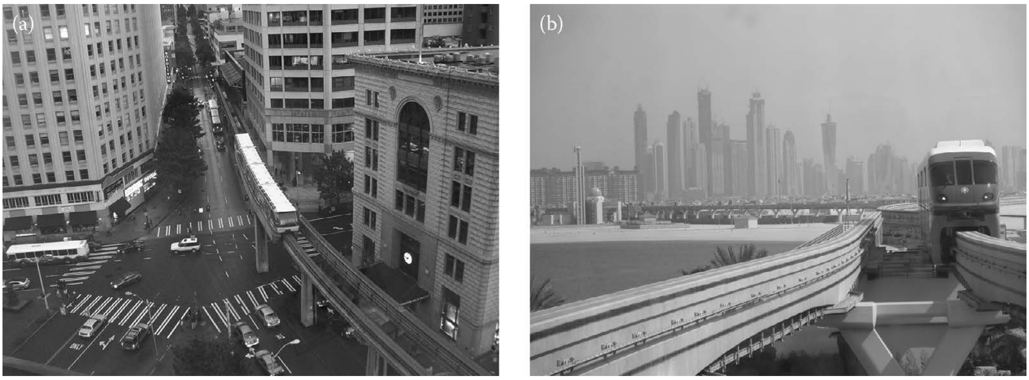


Figure 6.1 Straddled monorails: (a) In Seattle, USA. (Adapted from Fitzgerald, G. 2005, available online at: http://en.wikipedia.org/wiki/Seattle_Center_Monorail) (accessed 7 August 2015).) (b) In Dubai, UAE.

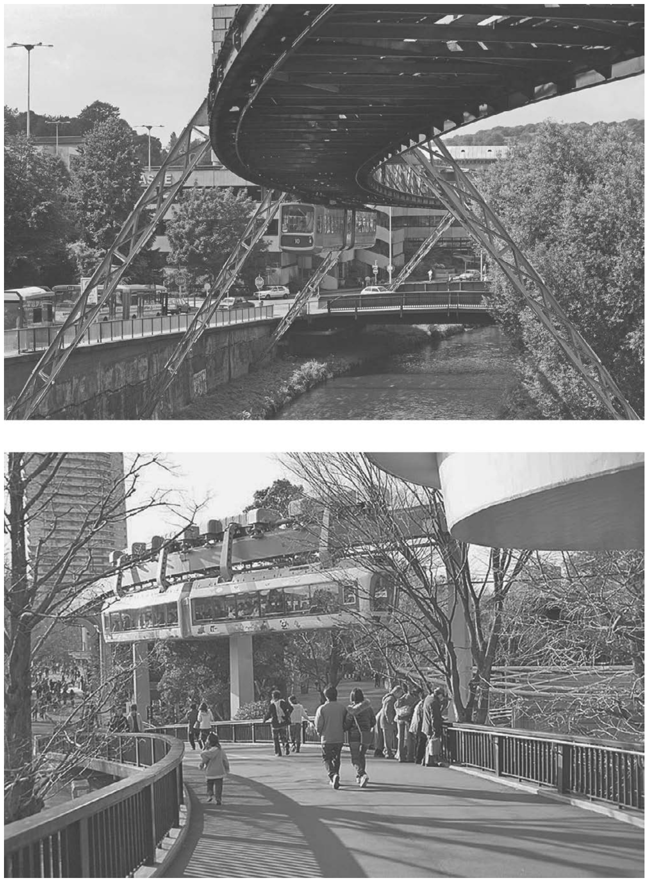
- *Cantilevered systems*: In the case of cantilevered monorails, the opposing moving trains share the same large-width beam. The trains balance with the aid of rubber-tyred wheels on surfaces that lie on the side ends of the beam (Figure 6.4). This technique has not yet been applied in practice.

6.2.2 Transport capacity

According to their transport capacity, straddled monorails can be classified into three categories (Kato et al., 2004)

- Small monorails
- Large monorails
- Standard (compact) monorails

Figure 6.2 Guideway of straddled monorail in Russia, Moscow. (Adapted from Lutex, 2009, available online at: en.wikipedia.org/wiki/Moscow_ Monorail (accessed 7 August 2015).



(a)

(b)

Figure 6.3 Suspended monorails (a) in Wuppertal, Germany (Photo: A. Klonos.) and (b) in the Ueno Zoo, Japan (Ueno Zoo Monorail Taito-ku, 2005).

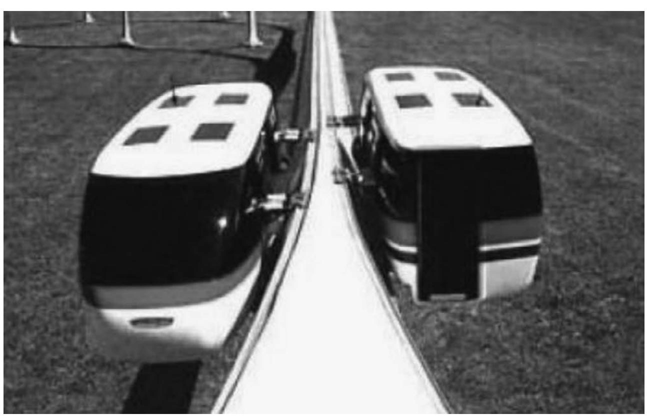


Figure 6.4 Cantilevered monorail system. (Adapted from Online image. Available at http://www.monorails.org/tMspages/TPindex.html (accessed 11 March 2015).)

In small straddled monorail systems, the width of the guidebeam (660–700 mm), the width of the vehicles (2.30–2.64 m) and the axle load (8 t) are relatively small. The train is composed of two carriages, each of which has a capacity of 60–80 passengers and can carry, considering a headway of 5 min, approximately 2,000 passengers/h/direction (the dimensions and the transport capacity of the vehicles differ according to the manufacturer).

In large straddled monorail systems, the width of the guidebeam (850–900 mm), the width of the vehicles (3.0 m) and the axle load (10–11 t) are relatively larger. The train is formed with 5–6 carriages, each of which has a transport capacity of 100–170 passengers and can carry, considering a headway of 5 min, approximately 12,500 passengers/h/direction.

In compact straddled monorail systems, the train is composed of 3–4 carriages, each of which has a transport capacity of 60–100 passengers and can carry, considering a headway of 5 min, approximately 4,800 passengers/h/direction.

6.2.3 System techniques

During the application and the development of monorails, various guidance techniques were used: Lartigue, Alweg, Steel Box Beam, Inverted T (straddled systems), Safege, I-beam and Double-flanged (suspended systems).

Figures 6.5 and 6.6 illustrate the techniques that are used for monorail systems.

6.3 CONSTRUCTIONAL AND OPERATIONAL CHARACTERISTICS OF THE SYSTEM

6.3.1 Permanent way

The permanent way is almost always elevated, and usually consists of two guidebeams (one beam per traffic direction 'double track'). Hence, the superstructure of monorail systems

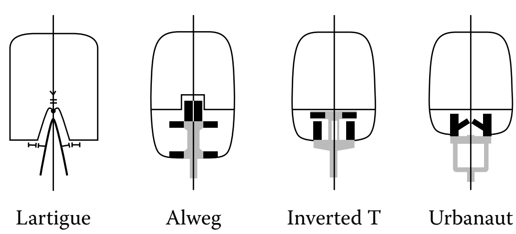


Figure 6.5 Straddled monorail systems – Guidance techniques. (Adapted from Online image. Available at www.urbanaut.com (accessed 11 March 2015).)

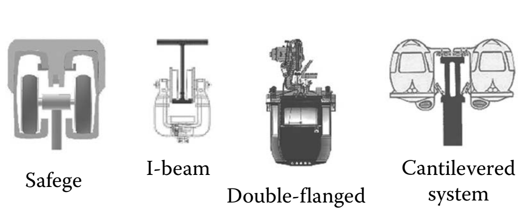


Figure 6.6 Suspended and cantilevered monorail systems – Guidance techniques. (Adapted from Online image. Available at http://www.monorails.org/tMspages/TPdoub.html (accessed 11 March 2015); Online image. Available at http://www.monorails.org/tMspages/TPmbeam.html (accessed 11 March 2015) and Online image. Available at http://fr.wikipedia.org/wiki/Monorail (accessed 11 March 2015).)

Figure 6.7 Special frame structure (straddle bents) used for the Okinawa monorail system in Japan. (Adapted from Monorail Society/Pedersen, K. no date. Okinawa Monorail, another Monorail Society Ex-clusive! available online at: http://www.monorails.org/tMspages/Okinawa1.html (accessed 13 July 2015).)

comprises of two beams with sufficient distance between them, so that the two opposite moving trains can cross paths (Figures 6.1 through 6.3).

A few monorail systems dispose only one guidebeam (single track) operating either in a loop (amusement park monorails) or by employing bypasses at stations so that a single beam can be used for bi-directional operation (Shonan suspended monorail) (Kennedy, no date).

The beam is usually made of concrete and, in some cases (e.g., Aerobus system) it is made of steel. Its width is approximately one-fourth of the vehicle's width.

When there is road traffic underneath the track, the minimum recommended height clearance is 5 m. However, for aesthetical reasons, there is a tendency to use much higher pillars, usually around 10–12 m high. In order to reduce the visual intrusion and allow for larger spans, a very effective technique is to use arc-shaped hunched girders for guidance. The span of the steel or concrete pillars ranges from 15 to 24 m.

In cases where the traffic volume of road vehicles under the monorail infrastructure is high, the pillars can be replaced by a special frame structure (straddle bents, Figure 6.7) (Monorail Society/Pedersen, no date).

As regards the track geometry alignment design, the minimum radius of the horizontal alignment is $R_c = 70$ m for large monorails and $R_c = 45–40$ m for small monorails ($R_c = 40$ m, at the depot), whereas the minimum radius of the vertical alignment is $R_v = 500$ m. The longitudinal gradients are up to 10% (the Lottle World system in South Korea and the monorail in Magdeburg have an $i_{max} = 20\%$ gradient).

In order to reduce the effect of the centrifugal force at horizontal curves, the beam bends not only at the horizontal level but also at the vertical level (development of cant).

The line length of monorail systems worldwide usually range between S = 1.5 and 12 km, while the distance between successive stops is usually longer than the respective distance for other urban means of transport (800–1,500 m). The longest straddled monorail network in the world is the Chongqing monorail (China, 74.7 km). One of its two lines has a length of 55.5 km and is the longest monorail corridor worldwide. The longest suspended monorail line worldwide is in Japan (Chiba City, 15.2 km).

The switching is achieved by one of the following four techniques (Pedersen, no date):

1. Segmented switch: The segmented track allows the beam to go from a straight position to a curved one.
2. Suspended monorail switch: It is applied to the suspended systems. Safege has pivoting horizontal plates inside the box beam that act as running surfaces in either direction

Figure 6.8 Switching diverters – Shonan monorail, Japan. (Adapted from online image available at: https://en.wikipedia.org/wiki/Shonan_Monorail, 2005.)

of the switch (Figure 6.8). Siemens (SIPEM) uses technology in which a vertical plate pivots inside the switch.

3. Beam replacement switch: A straight section of beam pivots to the side, while a curved section moves into place.
4. Rotary switch: Common to people-mover class, steel-beam monorails and rotating switches replace a straight section of track with a curved one.

Owing to the weight and size of the guidebeams, the process of switching is much slower in relation to the respective process for the conventional railway (12–20 s against 0.6 s).

6.3.2 Rolling stock

The trains are articulated. The maximum running speed is $V_{max} = 60–90$ km/h, and the acceleration/deceleration values range between 1.0 and 1.2 m/s^2.

The vehicles run on either four-axle or two-axle bogies using air suspension. The maximum axle load is $Q_{max} = 10–11$ t.

The vehicles' width ranges between 2.30 and 3.00 m according to the type of the monorail (small, large and standard).

The motors are fed by two electrical lines (bus bars), placed laterally to the guidebeam. The triple-phase current of 500 V converts to 1,500 DC or 750 DC, and with the aid of an electronic voltage regulator, the smooth acceleration and deceleration of the train is achieved. An emergency generator, installed in the central station, takes over the system's supply in the case of a power failure, so that the vehicles can be driven to the nearest stations. Owing to the elevated permanent way, it is possible to use solar panels along the guidebeam in order to gain energy for the power supply (Marconi Express Bologna).

Owing to their rubber-tyred wheels, the vehicles are sensitive to ice and snow. As a result, heating of the beam could be required, resulting in an increase in energy consumption. On the other hand, the vehicles move quietly without causing 'electromagnetic pollution'.

The transport capacity of a standard monorail train with four vehicles (4 persons/m^2) often ranges between 200 and 400 passengers (seated and standing).

6.3.3 Operation

The commercial speed is usually $V_c = 20–40$ km/h. The headway ranges between 3 and 15 min (minimum headway: 1.0 min), whereas the transport volume capacity of the monorail system depends on the size and the headway. In theory, maximum transport volume capacities of 20,000–25,000 passengers/h/direction can be achieved (trains of 600 seats scheduled every 1.5 min).

The track requires very low maintenance, and the rubber-tyred wheels are replaced approximately every 160,000 km (Kennedy, no date).

The elevated guideway imposes difficulties in evacuating the trains in case of emergency. Suspended monorails often have their vehicles' doors at the floor level, connected with stairs or slides, as in the case of aircrafts.

In straddled systems, the evacuation can be performed

- From the front or the rear vehicle, in a standing train, provided that the trains are articulated and the free movement of passengers along the train is feasible
- Laterally with the aid of a standing train approaching on the second guidebeam (side evacuation)
- With the aid of a stair, accessed from the ground
- With an emergency passenger platform (Las Vegas system, Figure 6.9)

Monorail systems move either with a driver or automatically (driverless systems). The use of the DTO (driverless train operation)/ATO (automatic train operation) technique at monorail systems is increasing in order to reduce the operational cost. The first fully automatic unmanned monorail was constructed in Japan, and commenced operation during the Osaka Expo in 1970.

Figure 6.9 Emergency walkway on Las Vegas Monorail, USA. (Adapted from Mikerussell at en.wikipedia. 2007, online image available at: https://en.wikipedia.org/wiki/Bombardier_Innovia_Monorail#/media/File:Monorail_incoming.jpg (accessed 8 August 2015).)

The total implementation cost of a double-track monorail system (totally elevated permanent way) ranges between €30 M and €90 M per track-km (2014 data), depending mainly on the transport capacity (www.lightrailnow.org/myths/m_monorail001.htm, https://pedestrianobservations.wordpress.com/2013/08/24/monorail-construction-costs/, and www.monorails.org/tMspages/HowMuch.html).

The cost of large-type monorails is almost double that of the small type.

6.4 ADVANTAGES AND DISADVANTAGES OF MONORAIL SYSTEMS

6.4.1 Advantages

- It occupies a small footprint for the pillars; thus, the expropriation costs are low. The guideway does not interfere with the existing road transport infrastructure. Especially when it is used at zoos, animals are protected from run overs.
- It does not pollute the atmosphere and moves almost noiselessly.
- It offers a panoramic view for its passengers.
- The construction time is relatively low (construction rate is approximately 8 km/year).
- The rubber adhesion allows higher acceleration, sharper curves and steeper grades than the metro.

6.4.2 Disadvantages

- It can only serve small distances.
- Its transport volume capacity is relatively low, although flexible modular transport system capacities are possible.
- It does not allow for a direct connection with another railway system (system incompatibility with other systems).
- The evacuation of the trains and the removal of passengers in case of an accident or immobilisation of the train on the track, are difficult (especially in the case of suspended systems).
- The infrastructure and the rolling stock are often constructed by different manufacturers, whose techniques may be incompatible with each other. Often, rolling stock manufacturers conclude agreements with infrastructure providers.
- The system causes visual intrusion due to its elevated permanent way.
- The implementation cost is almost three times higher than the implementation cost of the tram.
- Access to the boarding platforms is difficult due to the elevated stations.

6.5 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

Generally, the monorail is selected as a means of transport under the following circumstances:

- When there is need for a transport mode that will serve movement within amusement parks, zoos and so on
- For the transportation of passengers over small distances, and at areas that are particularly interesting in terms of their view
- For the connection of urban areas of the same altitude, where there is a natural barrier hindering their connection (e.g., water)

- Finally, at environmentally sensitive areas where there is great demand for transportation and it is not feasible to integrate a surface railway system

In recent years, monorails are increasingly used not only for recreational purposes, but also for urban transport, for serving connection with airports, movement within shopping malls and so on.

6.6 HISTORICAL OVERVIEW AND PRESENT SITUATION

6.6.1 Historical overview

The first monorail system was constructed in Russia in 1820 by Ivan Elmanov. Efforts for the development of single-beam railways, as an alternative to conventional railways, started in the beginning of the nineteenth century.

One of the first monorail systems that was developed was that of the French engineer Charles Lartigue, who constructed a line between Ballybunion and Listowel in Ireland in 1888. This line closed down in 1924 because it was ruined during the Irish Civil War. The Lartigue system uses a central single rail for support and movement, and two rails, placed at a lower level on both sides of the central rail, for guidance.

During the period 1900–1950, several systems were investigated, and were either abandoned at the design phase, or remained as prototypes, without ever being developed. In 1901, the Behr system was proposed for development between Liverpool and Manchester. In 1910, the Brennan system was also proposed for use in a coal mine in Alaska.

The first suspended monorail system was constructed in Wuppertal in 1901 (Figure 6.3a). The technique that was applied is that of the 'double-flange' (double-flanged, Figure 6.6). This system is the oldest monorail system in operation worldwide.

From 1950 to 1980, monorails were constructed in Japan (1957, Uena zoo, Figure 6.3b), in Disneyland of California (1959), in Seattle (1962), in Walt Disney World in Florida (1971), in Hawaii (1976) and in other areas. However, the use and application of monorails was very limited, as it was for all public transport systems due to their competitiveness with the private car. The guidance techniques that were used during this period were the Alweg technique for straddled systems and the French technique Safege for suspended ones (Figures 6.5 and 6.6).

Since the 1980s and up until today, the interest in the use of monorail systems has revived due to traffic congestion and urbanisation.

Figure 6.10 illustrates the historical evolution of the launching of new monorail systems. In the time period 1980–1990, there was an increase in the construction of new monorail systems which continues until today.

The most recently launched monorail systems are Sao Paolo's monorail in Brazil (August 2014), and the monorail system in the city of Xi'an, in China (30 January 2015). The most recently closed monorail systems are the monorail of Sydney, Australia (30 June 2013), and the monorail of Magdeburg, Germany (12 April 2014).

6.6.2 Present situation

There are 46 monorail systems in operation worldwide, of which 21 are in Asia, 10 in Europe, 13 in America and 2 in Oceania (Australia) (Figure 6.11). At the country level, most operational systems are located in the United States (10) and in Japan (9). Thirty-nine out of the 46 systems (i.e. approximately 85%) are straddled.

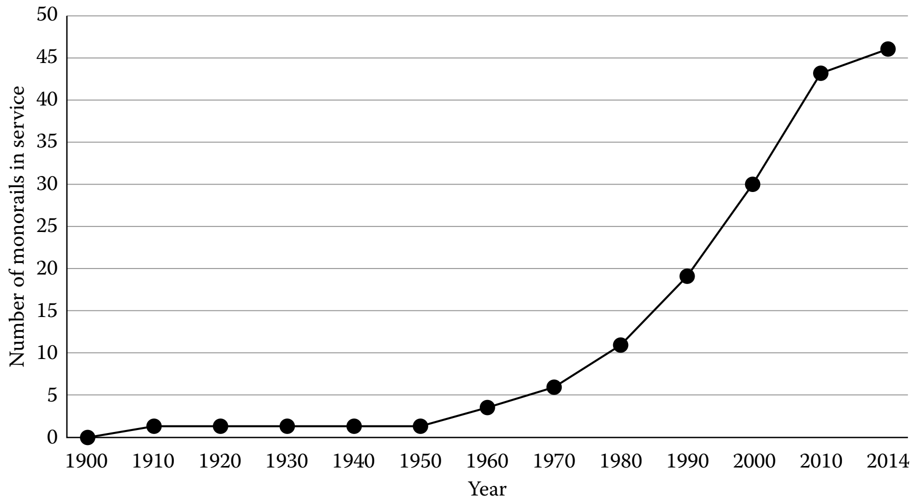


Figure 6.10 Historical evolution of monorail system's implementation.

All the above data and the data recorded are analysed in the following relate to the year 2014. The raw data were obtained both per country and per line, from various available sources and cross-checked. Afterwards, they were further manipulated for the needs of this chapter.

The total length of monorail lines is approximately 320 km.

Figure 6.12 illustrates the distribution of monorail systems according to their operational characteristics. As shown in this figure, the main use for monorail systems is the service of tourists at thematic parks, recreational areas and so on (43.5%), whereas their use as an urban transport mode is increasing in the recent years, amounting to 30.5%.

According to 2014 data, there are 13 monorail systems that either are under construction or are expected to start construction in 2015 (Table 6.1).

There is a tendency for fully automated systems. The monorail of Sao Paolo operates using the DTO/ATO system; such a system will also be applied in Riyadh, Saudi Arabia.

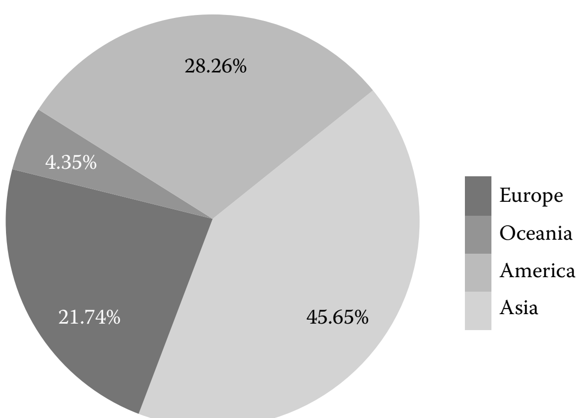


Figure 6.11 Distribution of monorail networks per continent.

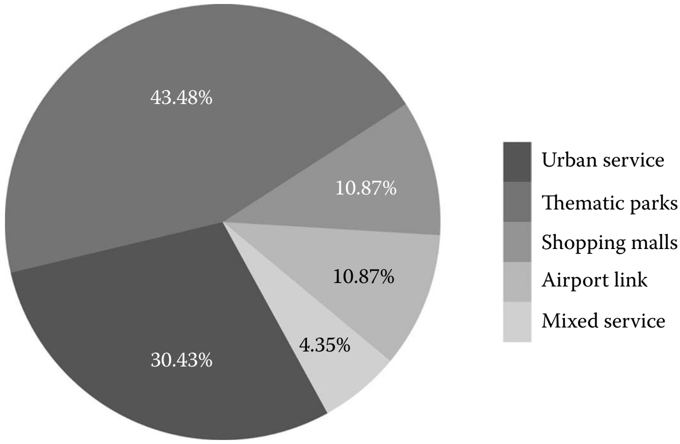


Figure 6.12 Distribution of monorail systems according to their operational use.

Table 6.1 Monorail systems under construction

	Country	*Name*	*Launch year (estimate)*	*Length (km)*	*Use*
1	Brazil	Manaus	–	20	
2	India	Kozhikode	2015	14	Park
3	Indonesia	Jakarta	2016	29	Urban service
4	Iran	Kermanshah	2015	13.5	Urban service
5	Iraq	Qom	2015	6.2	Urban service
6	Italy	Marconi Express Bologna	2015	5	Airport link
7	Nigeria	Port Harcourt	2015	2.6	Urban service
8	South Korea	Chongqing	2015	32	Urban service
9	South Korea	Daegu	2015	24	Urban service
10	China	Beijing	2015	24	Urban service
11	China	Xian	2015 (operates since 30 January 2015)	9.6	Urban Service
12	Vietnam	Da Nang	2015	3.0	Park
13	Saudi Arabia	Riyadh	2015	3.6	Shopping mall

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https://pedestrianobservations.wordpress.com/2013/08/24/monorail-construction-costs/ (accessed 11 March 2015).

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Mikerussell at en.wikipedia. 2007, online image available at: https://en.wikipedia.org/wiki/Bombardier_Innovia_Monorail#/media/File:Monorail_incoming.jpg (accessed 8 August 2015).

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Online image. Available at http://fr.wikipedia.org/wiki/Monorail (accessed 11 March 2015).

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Pedersen, K. no date, *The Switch Myth*, Report and online images available at http://www.monorails.org/tmspages/switch.html (accessed 11 March 2015).

Ueno Zoo Monorail Taito-ku, 2005, available online at: https://commons.wikimedia.org/wiki/File:UenoZooMonorail1280.jpg (accessed 7 August 2015).

www.lightrailnow.org/myths/m_monorail001.htm (accessed 11 March 2015).

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Chapter 7

Automatic passenger transport railway systems of low- and medium-transport capacity

7.1 DEFINITION

The means of transport classified into this category operate in an exclusive corridor by using, either individual vehicles, with a capacity of 3–25 persons, or low- and medium-transport capacity trains (50–250 persons, standing and seated). They are automated (driverless systems) and their rolling system includes at least one iron element (steel wheels on rails or rubber-tyred wheels on steel guideway). They belong to the general category of automated transport modes on 'fixed right-of-way' (automated guideway transit, AGT), which also includes higher capacity systems (e.g., driverless metro, monorail) (ACRP Report 37, 2010; Wikipedia, 2015c).

Depending on the traction system, they are classified into two categories (RPA, 2012):

- Cable-propelled systems
- Self-propelled electric systems

7.2 CABLE-PROPELLED RAILWAY SYSTEMS

7.2.1 General description and classification

The cable-propelled railway transportation systems uses vehicles propelled or hauled by cables. They move using either rubber-tyred wheels or steel wheels, moving on conventional rails, on steel beams of type I profile, on guideway made of reinforced concrete, or finally, on steel truss elements. They are classified into two main categories:

1. Systems aimed to connect areas of very high altitude difference. They are characterised by high longitudinal slopes (usually $i > 10\%–15\%$). The funicular, the cable railway and the inclined elevator belong to this category. These systems are investigated in detail in Chapter 10.
2. Systems aimed to serve connections of slighter longitudinal slope ($i < 10\%–15\%$) (Andréasson, 2001).

In this chapter the second category (II) is investigated exclusively. These systems fall under the category of automated people movers (APMs). They serve trips within small or medium distances (from $S = 300$ m up to $S = 12{,}000$ m) and their transport capacity can reach up to 8,000 passengers/h/direction (for distances up to 4 km). The running speed is $V_{max} = 35–50$ km/h.

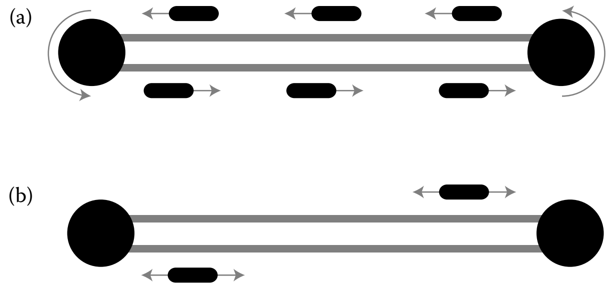

Figure 7.1 Cable-propelled railway transportation systems. (a) Detachable systems (continuously circulating configuration). (b) Non-detachable systems (shuttle-based configuration). (Adapted from Dale, S. 2014a, *Cable Cars, Lesson* 1: *Introduction*, available online: http://gondolaproject.com/2010/07/09/cable-cars-lesson-1-introduction/2014 (accessed 7 April 2015).)

Depending on whether the vehicles can be detached from the pulling cable or not, they are distinguished into detachable (continuously circulating configuration) and non-detachable systems (shuttle-based configuration) (Figure 7.1) (Dale, 2014a–d; Wikipedia, 2015a).

On the basis of their transport capacity, they are classified into two categories: cable trains and cable cars (individual vehicles) (Figures 7.2 and 7.3).

The cable train is composed of 2–6 vehicles. The vehicles are attached to a steel pulling cable and are interconnected. The train accelerates and decelerates through the pulling cable. Depending on the configuration of the superstructure, the trains do not change pulling

Figure 7.2 Cable train. (Adapted from Wikipedia. 2015b, *Cable Liner,* available online: http://en.wikipedia.org/wiki/Cable_Liner,2014 (accessed 7 April 2015).)

Figure 7.3 Cable car. (From Doppelmayr, 2015.)

cables at the terminal or intermediate stations (non-detachable systems), or they have the ability to change cables (detachable systems).

The cable cars (many small individual vehicles) are pulled through a constantly moving cable. The vehicles do not accelerate or decelerate via the pulling cable as in the previous case. Upon arrival at the station, they are detached from the pulling cable and the deceleration system of the station is activated (decelerator–conveyor–accelerator system). After the completion of passenger boarding and alighting, the acceleration system of the station is activated, guiding the vehicles out of the station. Through the acceleration system of the station, the speed of vehicles is equalised with the speed of the pulling cable, allowing thus, without being noticed by the passengers, the smooth reattachment of vehicles with the pulling cable. This system has low dwell time ($t_s < 90$ s) and is suitable when there is a constant flow of passengers.

7.2.2 Constructional and operational features of the systems

7.2.2.1 System 'principles' and superstructure configurations

The operation of cable-propelled railway systems may be performed by three different 'principles': shuttle, loop and continuous movement (ACRP Report 37, 2010; Wikipedia, 2015a).

7.2.2.1.1 Shuttle 'principle'

The shuttle 'principle' does not allow the detachment of the train from the cable; it is used for small route lengths (up to 3 km) and is characterised by relatively high dwell time. The transport capacity of the system depends on the length of the system and the number of intermediate stations, and may reach up to 6,000 passengers/h/direction.

Figure 7.4 illustrates the various possible superstructure configurations in case of operation by applying the shuttle 'principle'.

In case (a) (single-lane shuttle), the system operates with only one train, which runs between the two terminals, along the single track without any bypass area, by using just one pulling cable. The total number of stations amounts to two (the terminals), but it is also

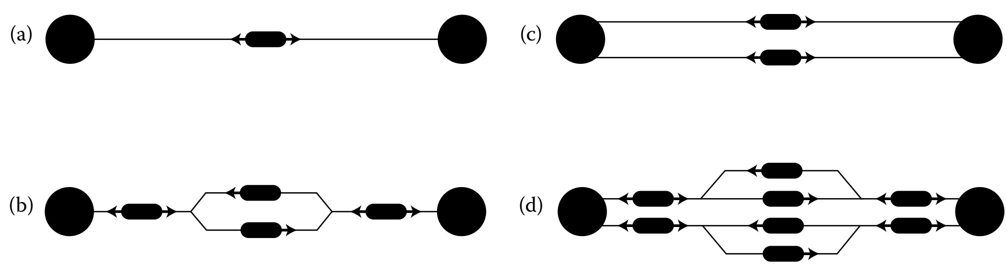

Figure 7.4 Various possible superstructure configurations in the case of operation of cable-propelled railway systems with the shuttle 'principle.' (a) Single-lane shuttle, (b) single-lane shuttle with bypass, (c) double-lane shuttle and (d) double-lane shuttle with bypass. (From Lea+Elliott Inc., 2015.)

possible to set up to three intermediate stations. This simple superstructure configuration is preferred in cases of low-demand transport work.

In case (b) (single-lane shuttle with bypass), the system operates with two trains, running among the terminal stations, along a single track with a bypass area, by using either one or two pulling cables (Dale, 2014d).

The first subcase (one pulling cable) allows for very steep longitudinal slopes and is used in funiculars (see Chapter 10). The bypass area is set necessarily in the middle of the route and the intermediate stations (0–3) are located at symmetrical distances.

The second subcase (two pulling cables) allows for the bypass area to be set on a different point, other than the middle of the route. This configuration enables the operation of one of the two trains, by providing 24-h service (Figure 7.5).

The configuration of case (c) (dual-lane shuttle) operates with two trains, running among the terminal stations, along a double track without any bypass area, by using two separate pulling cables. It is possible to operate the system with up to three intermediate stations.

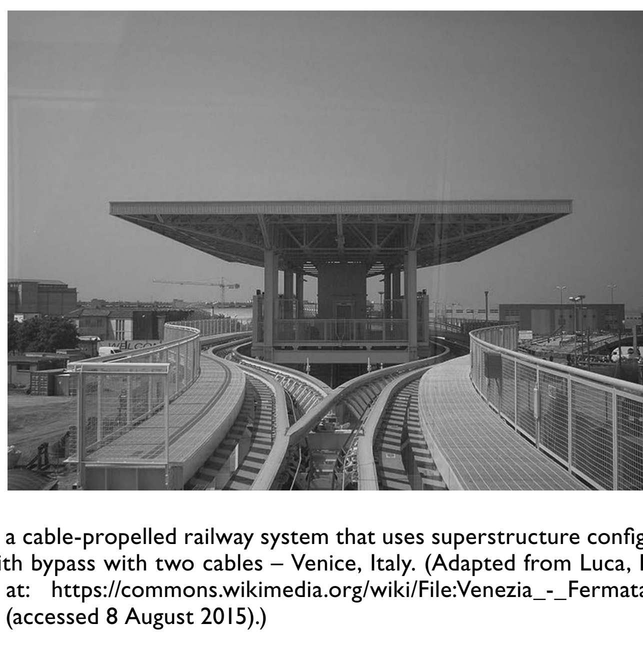

Figure 7.5 Station of a cable-propelled railway system that uses superstructure configuration of single-lane shuttle with bypass with two cables – Venice, Italy. (Adapted from Luca, F. 2010, Online image available at: https://commons.wikimedia.org/wiki/File:Venezia_-_Fermata_Marittima_people_mover.jpg (accessed 8 August 2015).)

This configuration ensures a high level of service, while outside rush hours only one train can operate as in the case of single-lane shuttle.

Finally, the configuration of case (d) (dual-lane shuttle with bypasses) operates with four trains. It consists of two parallel single-lane shuttles with bypass, doubling the system performance and providing a high level of service, while outside rush hours it can operate only with one train as in the case of single-lane shuttle with bypass.

7.2.2.1.2 Loop 'principle'

The loop 'principle' allows for train detachment from the cable, and at the same time serves a large number of intermediate stations, as well as the simultaneous use of many trains.

In Figure 7.6, the three possible superstructure configurations (single loop, double loop and pinched loop) are displayed in the case of operation of cable-propelled railway systems with the loop 'principle'.

The loop can be single (Figure 7.6a) or double (Figure 7.6b).

In a single-loop case, due to the movement of trains toward one direction only, the level of service is low, since passengers who wish to move to the previous station are forced to run throughout the system, in order to reach their destinations.

In the double-loop case, this issue does not exist, as the movement of trains is possible in both directions (Figure 7.7). This possibility allows for maintenance procedures and non-interruption of system operation in case of a temporary problem related with one of the two loops. The double-loop configuration holds double transport capacity compared with the single-loop configuration.

The configuration of pinched loop is similar to that of dual shuttle, with the difference that the track change equipment is required at terminal stations (turntable). This layout minimises dwell times, as it allows for the simultaneous movement of more than three trains along the track. This design uses several haul rope loops, which adjoin and overlap one another at stations. Switching the trains between the haul ropes creates a synchronised, circular flow of trains around the system.

By using the loop 'principle', dwell time is lower than the shuttle 'principle', and is determined by the length of the biggest loop of the pulling cable. Zero to seven stations, and two to three haul rope changeover stations, can be sited, achieving route lengths of up to 12 km.

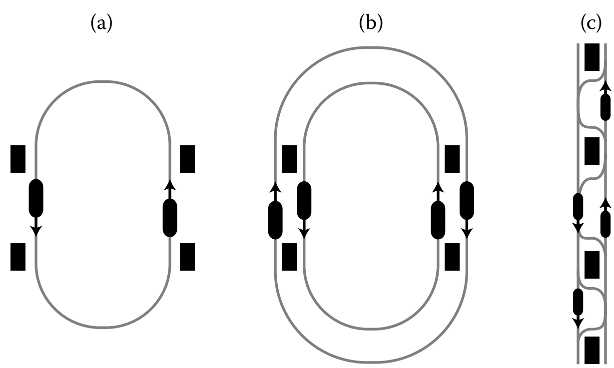

Figure 7.6 Possible superstructure configurations in the case of operation of cable-propelled railway systems with the loop 'principle.' (a) Single loop, (b) double loop and (c) pinched loop. (From Lea+Elliott Inc., 2015.)

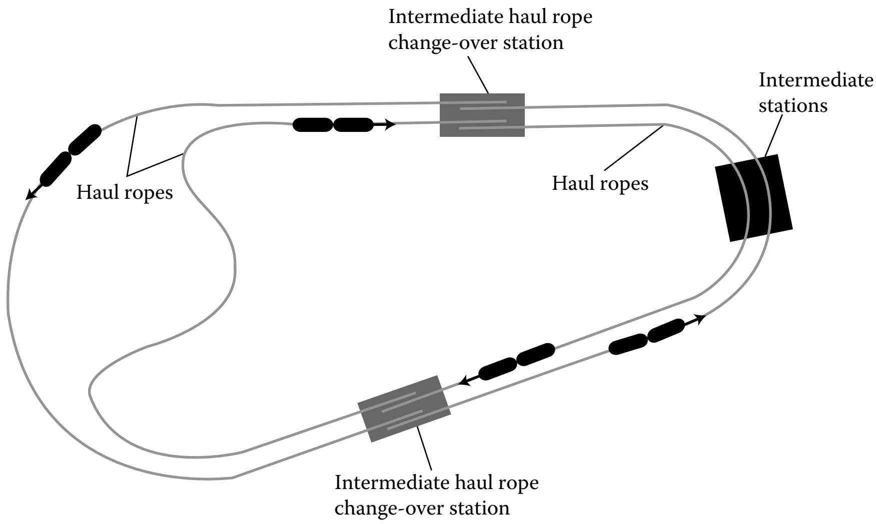

Figure 7.7 Superstructure configuration in the case of double-loop 'principle.' (From Doppelmayr, 2015.)

Transport capacity is almost independent of the system's length and the number of stations, and is defined by the train attached to the longest pulling cable. It can reach up to 6,000 passengers/h/direction.

7.2.2.1.3 Continuous movement 'principle'

The continuous movement principle, in contrast to the shuttle layouts, requires two separate tracks, since vehicles are detachable from the cable (Figure 7.8). It operates on individual vehicles (cable cars). It is characterised by exceptionally low dwell time, less than 1 min, regardless of the system's length. It allows for setting up of up to 13 intermediate stations, and 2–3 haul rope changeover stations, achieving long system lengths (approximately up to 9 km). Possible superstructure configurations are of linear shape (straight layout), with

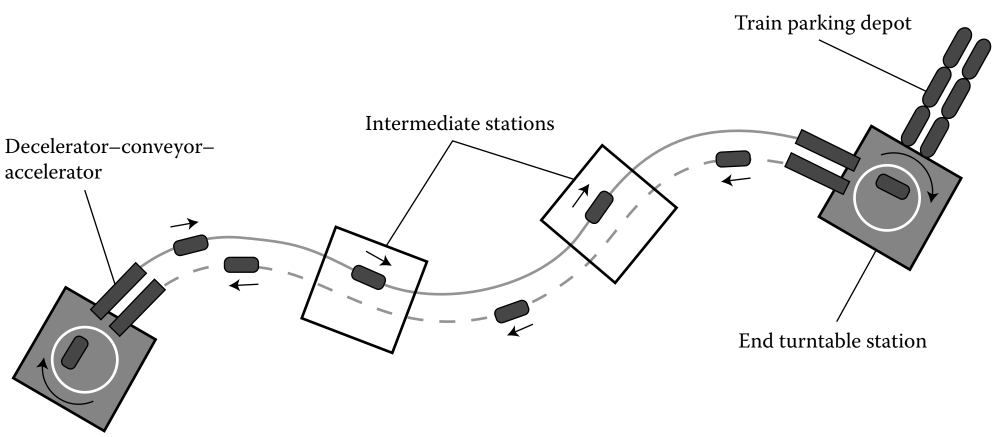

Figure 7.8 Continuous movement 'principle' – linear shaped-superstructure configuration. (From Doppelmayr, 2015.)

circular single and double loops. The transport capacity of the system is independent of its length and the number of stations, and can reach up to 4,500 passengers/h/direction.

7.2.2.2 Guideway

The guideway of cable-propelled railway systems can be integrated into the surrounding area either at ground, under the ground or on an elevated structure.

In elevated systems, the guideway is normally implemented on steel beams (coupled) in truss form, which are placed on supports made of reinforced concrete (Figure 7.9). Tracks do not require heating in case of low temperatures, and moreover, the use of variable spacers between the coupling and the supports allows for easy counterbalancing of any differential settlement.

They can also use guideway made entirely of reinforced concrete.

Platforms are designed in order to be placed on the same level with the floor of the vehicles, facilitating in this way, passenger boarding and alighting. Platform length is defined based on the number of vehicles of each train. For the safety of passengers, the station platform is separated by sliding doors from the traffic corridor.

In Figure 7.10, a schematic representation of the rolling system of the cable-propelled railway systems (cable-driven people movers) is displayed and in Figure 7.11 the pulling cable under the vehicles is shown.

In Figures 7.12 and 7.13, the sheaves that carry on the cable along the straight segments of the track and guide it in curved track sections are illustrated.

7.2.3 Advantages and disadvantages

Advantages

- Environmentally friendly, since they have low energy consumption and zero gas emissions.
- Highly reliable, to a percentage of more than 99.5%, as their operation is not affected by extreme weather conditions (strong winds, snow and very high temperatures).
- Systems running on an elevated structure are adjusted to soil settlement, due to low total weight (vehicles + superstructure).

Figure 7.9 Steel guideway of cable-propelled railway systems, supported by pillars made of reinforced concrete. (From Doppelmayr, 2015.)

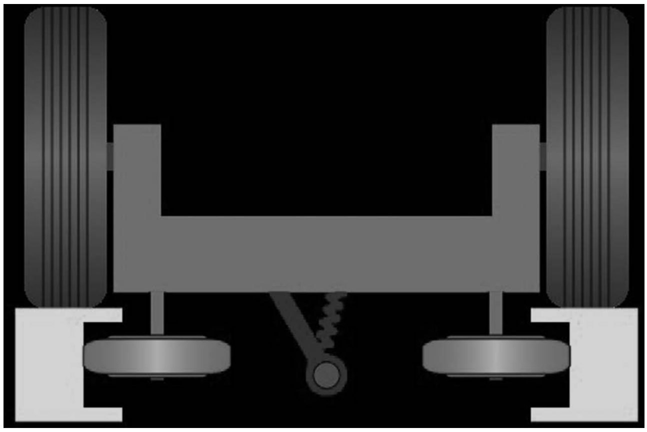

Figure 7.10 Schematic representation of the rolling system of cable-propelled railway systems. (Adapted from Wikipedia. 2015d, *Venice People Mover*, available online: en.wikipedia.org/wiki/Venice_People_Mover (accessed 7 April 2015).)

Figure 7.11 Pulling cable of cable-propelled railway systems. (Adapted from Doppelmayr. 2011, Online image available at: http://en.wikipedia.org/wiki/Cable_Line (accessed 8 August 2015).)

Figure 7.12 Guidance sheaves of cable-propelled railway systems. (From Doppelmayr, 2015.)

Figure 7.13 Guidance sheaves of cable-propelled railway systems. (From Doppelmayr, 2015.)

- Maintenance works are comparatively smaller and no facilities are required, to be set outside the line. Maintenance works are normally performed in a special area below the platform level.
- All the advantages of an automated transport system (reliability, safety and low operational cost).

Disadvantages

- Continuous rolling noise due to the movement of the cable, which causes annoyance when the system is integrated into densely populated areas.
- Non-expandable systems.
- Limited length and transport capacity.
- The presence of many curved segments increases energy consumption due to friction in guidance sheaves.

7.2.4 Requirements for implementing the system

Cable-propelled railway systems are chosen to usually serve movements within airport areas, at big hotel complexes, casinos, conference and health centres, and in areas of educational or corporate campuses.

In the urban environment, they may feed heavier railway transport systems, as extensions or by connecting their lines.

Cable-propelled railway systems of continuous movement are suitable for any application shorter than 8 km. Usually they serve public parking areas and hospitals, universities or shopping malls, or they connect central railway stations or metro stations with suburbs.

In some cases, such as in the city of Perugia, Italy, they serve purely urban movements (Figures 7.14 and 7.15). Known as the MiniMetro the Perugia system can operate so frequently that dwell time is almost non-existent. Extending to 3.2 km in length, the system currently has seven intermediate stations. Vehicles can be inscribed into horizontal alignment radii of up to 30 m and move in longitudinal slopes up to 15%. Vehicles

Figure 7.14 Cable-propelled railway system of Perugia, in Italy (MiniMetro). (From LEITNER AG/SPA, 2015.)

Figure 7.15 Guideway of the cable-propelled railway system of Perugia (MiniMetro). (From LEITNER AG/SPA, 2015.)

bear rubber-tyred wheels, and they are 5 m long with a maximum transport capacity of 50 passengers. In stations, vehicles are automatically detached from the pulling cable, and conveyed through the station by an independent conveyor system.

Table 7.1 displays the constructional and operational features of various cable-propelled railway systems of low- and medium-transport capacity that move on relatively slight longitudinal slopes.

Table 7.2 displays indicatively the total implementation cost (infrastructure + rolling stock) per length-km for some systems (MetroTram, 2014).

7.3 SELF-PROPELLED ELECTRIC SYSTEMS

7.3.1 General description and classification

These self-propelled railway systems, depending on their power supply system, are distinguished into two categories based on those using

- batteries
- outside power feeding

Systems that use lower-capacity vehicles (3–25 persons) belong to the first category. These vehicles are power supplied by lithium-ion or lead-acid batteries that provide autonomy in

Table 7.1 Technical and operational features of various cable-propelled railway systems of low and medium-transport capacity that move in relatively slight longitudinal slopes

Name		*Country/city*	*Track superstructure configuration*	*Starting year of operation*	*Line length (m)*	*Running speed (km/h)*	*Train headway (s)*	*Train transport capacity (persons)*	*Transport system capacity (passengers/h/direction)*	*Number of vehicles per train/number of trains*	*Number of stations terminals (intermediate)*
Mandalay Bay Tram	System I	USA	Dual-lane shuttle	1999	838	36	300	160	1,300	5/2	4(2)
	System II						220		1,900		2
Air–Rail Link		United Kingdom/ Birmingham	Dual-lane shuttle	2003	585	36	120	160	1,600	2/2	2
International Airport Link		Canada/ Toronto	Dual-lane shuttle	2006	1,473	43.2	250	196	2,150	6/2	3(1)
International Airport Shuttle		Mexico/ Mexico	Single-lane shuttle	2007	3,025	45	650	104	600–800	4–6/1	2
Tronchetto – Piazzale Roma Shuttle		Italy/Rome	Single-lane shuttle with bypass	2010	870	29	190	200	3,000	4/2	3(1)
MGM City Center Shuttle		USA/Las Vegas	Dual-lane shuttle	2009	650	37.8	150	132	3,000	4/2	3(1)
NDIA Shuttle		Qatar/Doha	Dual-lane shuttle	2013	500	45	110	190	6,000	5/2	2
Cabletren Bolivariano		Venezuela/ Caracas	Pinched loop	2012	2,100	47	270	212	3,000	4/4	5
Oakland Airport Connector		USA	Pinched loop	2014	5,100	50.4	280	148	1,900	4/4	3
Perugia Minimetro		Italy	Continuous movement	2008	3,200	36–43.2	90	25	<3,000	1/25	7

(*Continued*)

Table 7.1 (Continued) Technical and operational features of various cable-propelled railway systems of low and medium-transport capacity that move in relatively slight longitudinal slopes

Name	*Country/city*	*Track superstructure configuration*	*Starting year of operation*	*Line length (m)*	*Running speed (km/h)*	*Train headway (s)*	*Train transport capacity (persons)*	*Transport system capacity (passengers/h/direction)*	*Number of vehicles per train/number of trains*	*Number of stations terminals (intermediate)*
Cincinnati Concourse Train	USA	Dual-lane shuttle (underground)	1994	400		132		5,700	3/2	3(1)
Detroit Express Tram	USA	Single-lane shuttle with bypass	2002	1,100		192		4,000	2/2	3(1)
Minneapolis St. Paul Airside	USA	Pinched loop	2004	800		186		1,700	2/2	4
Minneapolis St. Paul Landside	USA	Dual-lane shuttle (underground)	2001	400		84		5,200	3/2	2
Tokyo Narita	Japan	Dual-lane shuttle with bypass	1992	300		108		9,800	1/4	2
Zurich Skymetro	Switzerland	Pinched loop (underground)	2003	1,100		150		4,500	2/3	2

Source: Adapted from ACRP Report 37. 2010, *Guidebook for Planning and Implementing Automated People Mover Systems at Airports*, Research sponsored by the Federal Aviation Administration, TRB Washington, DC.

Table 7.2 Indicative total implementation cost (infrastructure + rolling stock) of cable-propelled systems of low and medium transport capacity

Cable-propelled system	*Cost (€ M/length-km)*
Birmingham Air–Rail, UK	12.7
Oeiras Municipality (Lisbon West) SATUO, Portugal	19.5
Perugia Minimetro, Italy	32.0
Venice, Italy	26.5

Source: Adapted from MetroTram. 2014, Information portal about guided public transportation in Europe since 1980. *MetroTram*, available online: http://www.metrotram.it/index.php?vmcity=VENEZIA&ind=0&num=1&lang=eng&vmsys=fun (accessed 7 April 2015).

the region of about 60–75 km. Battery charging is performed, either in the platforms, or in a specially formed charging area, within the maintenance facilities of the system.

They serve short-distance movements of individuals or small groups without making intermediate stops. The vehicles launch from the starting point, either upon travel request, or based on scheduled itineraries (Bly and Teychenne, 2005).

They are classified into personal rapid transit (PRT) and group rapid transit (GRT) systems.

In PRTs (or podcars), the vehicles have a transport capacity of 3–6 persons and they move exclusively within a unique traffic corridor (Gilbert and Perl, 2007).

In GRTs, the vehicles have a higher transport capacity (20 persons) and they can be operated either in a unique traffic corridor or in a network.

Systems of medium-transport capacity belong to the second category, which are normally used for service in airport areas. They fall under APM systems.

7.3.2 Battery-powered systems

The concept and development of PRT/GRT systems started in the 1950s. The first systems were set in operation in Japan (CVS system) and in the United States (Morgantown system) in 1975, of which only the second one has kept on operating. During the decade of 2000–2010, a more systematic development essentially took place in PRT/GRT systems.

Most systems of this category run on rubber-tyred wheels in a guideway made of asphalt or concrete. On the basis of the criterion set in Section 7.1, with regard to the rolling system, these systems are not included in the railway transportation systems and are therefore not investigated further.

A number of systems that met the criterion of Section 7.1 and can be characterised as railway systems are currently under research, in the stage of prototype construction or in the stage of testing, and specifically the following: LINT/Modutram/Mexico, Eco Mobility/Poland, Cabinentaxi/Germany, Coaster/Austria, Tubenet Transit System/China, Skyweb Express Taxi 2000/USA, Cybertran/USA, ecoPRT/USA, Skycabs/New Zealand, BM Design/Finland and Techvilla/Finland.

A system that can be classified in this category is the Vectus/POSCO ICT (POSCO ICT Co., Ltd, 2015). This system can operate both with batteries (autonomy for distances up to 4–5 km) and power collecting shoe (3rd and 4th rail power feeding system). The system is characterised by short connection length, high running speeds (V_{max} = 60 km/h) and very small vehicle headways (4–10 s). Actually this system operates exclusively with outside power feeding (3rd and 4th rail), at the city of Suncheon, South Korea (see Table 7.3 and Figure 7.16).

The vehicles are equipped with an automatic obstacle tracking and detection system, which allows acceleration or immobilisation of vehicles in case of obstacle detection.

Table 7.3 PRT automated passenger railway system currently in operation (electric power feeding)

Name of system/ manufacturer	*Category of system*	*Country/city-starting date of operation*	*Features*
Vectus/POSCO ICT	PRT	S. Korea/Suncheon 4/2014	Route length: 4.64 km of elevated, bi-directional guide way Fleet: 40 vehicles Vehicle transport capacity: 6–9 passengers Max. speed: 35–50 km/h System transport capacity: 1,500 passengers/h/direction Power supply system: exclusively 3rd and 4th Rail Power Supply System DC 500 V Minimum vehicle headway: 5–6 s Rolling system: iron wheels made of polymer material running on rails

Source: Adapted from POSCO ICT Co., Ltd, 2015. SunCheon SkyCube PRT Project Overview v1.1.

Each vehicle is equipped with a two-way communication system between the passengers and the system control centre. Moreover, the vehicles are air conditioned and they have a closed-circuit monitoring system with cameras. There are LCD screens in passenger seats, which provide useful information during the trip.

The floor of the vehicles is usually located on the same level with the boarding and alighting platform.

Vehicles maintain a minimum running track headway time of 5–10 s to prevent collision, which is ensured through the closed-telecommunication system and automatic detectors.

Advantages (Wikipedia, 2015c)

- Higher transport capacity compared with private cars, due to the ability of small vehicle headways.

Figure 7.16 Vectus/POSCO ICT PRT automated passenger railway system, Suncheon, South Korea. (From POSCO ICT Co., Ltd., 2015.)

- All the advantages of an automated transport system (reliability, safety and low operational cost).
- Very low dwell time. Outside rush hours, the level of service is increased as there is always typically an available vehicle waiting at the starting point.
- Small right-of-way.
- Low travel time, since commercial speeds of PRTs are double than those of buses.
- Ability of exclusively private transport. Passengers choose whether they will travel with fellow travellers.
- Reduction of air pollution.

Disadvantages (Wikipedia, 2015c)

- Lower transport capacity compared with buses and light urban railway systems.
- High total implementation cost per length-km compared with buses, but much lower than the cost required for the construction of a light railway system (tram, monorail) (Indicative cost €8-11 M/length-km, double track, infrastructure + rolling stock). It is stated that the system at Heathrow (it uses rubber-tyred wheels on asphalt) has a cost of about €10 M/length-km (ATRA, 2014a) and the system at Suncheon €10 M/length-km (POSCO ICT Co., Ltd, 2014).

PRT systems operate essentially as feeder modes of heavier railway transport systems, as an alternative against other slower transport modes, such as buses, and also moving on foot.

They are chosen to serve normally movements in airport areas, within shopping centres, university or hospital, for local trips in new cities, and for the supply/distribution around the stations of railway systems.

7.3.3 Outside power feeding systems

The systems of this category are also characterised as APMs.

APM vehicles are electrically powered by either direct current (DC) or alternating current (AC) provided by a power distribution subsystem along the guideway. Vehicle propulsion may be provided by either 750 or 1,500 V DC rotary motors, 480 or 600 V AC rotary motors or by AC linear induction motors (LIM).

Self-propelled vehicles or trains run in an exclusive double-track corridor through rubber-tyred wheels or steel wheels on rails. Their maximum running speed fluctuates between V_{max} = 50 and 75 km/h.

They were initially developed to serve urban areas and then they were applied mostly in big airports. According to 2010 data, APMs were in operation in 44 airports worldwide (electric and cable-propelled in total) (ACRP Report 37, 2010). Their permanent way is integrated into the space, on surface, underground and in elevated cross sections.

The latest systems have been constructed in order to facilitate the movement of passengers and airport employees within the airport's security zone (movement between check-in [terminal] areas and aeroplane gates). These systems are characterised by high transport capacity (8,500–9,000 passengers/h/direction [passengers with carry-on baggage only]), high speed, low train headway (2 min) and longer connection lengths.

More recently, they have been used to connect terminal stations with areas outside the airport's security zone, such as parking areas, car rental services, hotels and other related employment and activity centres. These systems are characterised by lower transport capacity (3,000 passengers/h/direction, 50 passengers/vehicle [passenger all baggage]) and higher train headway (3 min).

Table 7.4 Technical and operational features of various electric automated railway systems of low- and medium-transport capacity – systems in operation in airports

Name	*Country*	*Track superstructure configuration*	*Starting year of operation*	*Line length (km)*	*Train headway (min)*	*Transport system capacity (passenger/h/direction)*	*Type of provided service*	*Number of vehicles per train/number of trains*
Gatwick Airport Transit	United Kingdom/London	Dual-lane shuttle, elevated	1987	1.2	2.6	4,200	Landside conveyance	3/2
Madrid Barajas Airport	Spain	Pinched loop, underground	2006	2.2	2	6,500	Airside conveyance	3/6
Miami International Airport	USA	Dual-lane shuttle, elevated	1980	0.3	2	6,750	Airside conveyance	3/2
New York–John F. Kennedy International Airport	USA	Pinched loop, primarily elevated	2003	13.0	2–4	3,780	Landside conveyance	1–2/32
Kansai International Airport	Japan/Osaka	Four single-lane shuttles with bypasses, elevated	1994	2.2	2–2.5	14,400	Airside conveyance	3/9
Paris Roissy Charles de Gaulle Airport	France	Shuttle, underground	2007	0.6	2	4,500	Airside conveyance	2/3
Washington Dulles International Airport	USA	Pinched loop, underground	2010	2.3	2	6,755	Airside conveyance	3/29

Source: Adapted from ACRP Report 37. 2010, *Guidebook for Planning and Implementing Automated People Mover Systems at Airports*, Research sponsored by the Federal Aviation Administration, TRB Washington, DC.

Self-propelled APM systems have high requirements as far as the infrastructure is concerned demanding heavy concrete structures for the guidway and heatings of the guideway running surfaces in winter. Thanks to guideway switches they can be operated in the pinched-loop mode. Self-propelled APMs are extremely reliable and reach availability rates of more than 99.5%.

Table 7.4 displays the constructional and operational features of various self-propelled electric railway systems in operation in airports.

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Chapter 8

Suburban railway

8.1 DEFINITION AND CLASSIFICATION OF SUBURBAN RAILWAY SYSTEMS

According to international practice, suburban railway usually comprises an electrical passenger railway transport system, whose features are adapted for commuting transportation service within the geographical boundaries of large urban agglomerations (suburbs and satellite regional centres) (Figure 8.1).

Its range can exceed 100 km and may even reach up to 150 km. The nomenclature varies. When covering lengths of 30–50 km, it is designated as commuter or urban rail, whereas when it covers greater lengths, it is sometimes called 'regional' railway.

It is defined by high-frequency services (usually trains run every 10–20 min), commercial speeds of $V_c = 45–80$ km/h and transport capacity of up to around 60,000 passengers/h/direction.

Suburban services started in Berlin, Germany, at the end of the nineteenth century. From then on, increasingly more cities have integrated the suburban railway into their transport system.

8.2 CONSTRUCTIONAL AND OPERATIONAL CHARACTERISTICS OF THE SYSTEM

The main characteristics of the suburban railway systems have been given in Table 1.6. In addition to those, the following should also be noted:

The track design speed of lines servicing exclusively suburban trains varies between $V_d = 120$ and 160 km/h.

The track must be double so as to achieve sufficient track capacity and to reduce travel time.

The suburban train runs basically at grade for most of the journey; however, it can also run underground (city centres) or elevated. The total length of the route requires fencing on both sides of the track.

To provide suburban services, electrical multiple units (EMU) are usually used. Apart from the railcars, loco-hauled trains and push-pull trains are also used. The total number of vehicles usually varies from 2 to 8, and it may fluctuate, depending on the transport demand.

As the route is relatively long in comparison with urban transit (metro, tramway), a higher level of comfort during the trip is required and seated accommodation is increased (Figure 8.2). More specifically, the percentage of seated passengers must reach 50% of the total number of available seats.

(a)

(b)

Figure 8.1 (a) Electric suburban railway, double-deck, Switzerland. (b) Electric suburban railway, EMU, Greece. (Photo: A. Klonos.)

Figure 8.2 Internal arrangement in vehicles of suburban trains, Russia. (Photo: A. Klonos.)

The vehicles must have a large number of external doors which ought to be wide so as to facilitate rapid boarding and alighting and reduce halt times at stops. When the commuting service is very busy, while the length of platforms cannot be easily increased, double-deck carriages may be used (Figure 8.1a).

The usual frequency for the service varies from 10 to 30 min. The lowest value of train headways is 1.5 min. The lowest acceptable frequency for a suburban train is one train per hour.

The use of a clock-face timetable (e.g., every xx.03 and xx.33 and so on) is advisable since it helps the passengers to easily memorise the departure times, while at major stations it is suggested to fix departure times at round minutes (e.g., 9:10 instead of 9:12).

The traction characteristics of the powered vehicles must be able to provide swift acceleration, and the train must be equipped with particularly efficient braking systems.

The transport capacity of the trains can vary between 250 and 1,500 passengers with carriages that have either moderate-high-floor or high-floor. The transport capacity of the suburban railway can reach 60,000 passengers/h/direction (trains at 1.5 min intervals, train transport capacity 1,500 passengers) with values varying usually between 5,000 and 40,000 passengers/h/direction.

The signalling and train protection system must allow short headways. In case suburban services are mixed with other types of services (e.g., long-distance passenger, freight), the requirements increase. In such cases, some stations should allow overtaking, and if the traffic volume is high, even quadruple tracks are used.

In order to serve demand during peak hours, there are various solutions. Their application presumes analytical data for the origin and destination of the passengers.

These solutions are

- *Skipping of stations*: Trains do not stop at every station. They are distinguished under categories (e.g., A, B, etc.), and each category serves particular stops. This method results in the increase of the commercial speeds of the trains. On the other hand, it creates some problems to the users, such as
  - The obligation to harmonise services for transfers from stops of one category to stops of another category
  - The complexity of the service system
- *The zonal timetabling method*: The difference from the previous method is that in this method the trains serve all of the stations of one or more designated areas, and then they are directed to the main station of the line (the terminal) without any stop. This method is suitable for cases where the demand is greater at a limited number of stations, instead of being evenly distributed throughout the line. Problems from the application of this method arise when there is demand for service among the zones that are not connected.
- *Return without passengers*: It is the method which is applied in cases where the demand is limited to one direction. In such cases, trains moving in the other direction are running empty, and with accelerated nonstop timetable in order to assume earlier duties along the more demanding direction.
- *Increase in the number of vehicles of the train formation – Exploitation with multiple units (MU)*: In this case, longer platforms are necessary.

8.3 ADVANTAGES AND DISADVANTAGES OF THE SUBURBAN RAILWAY

8.3.1 Advantages

The great development of the suburban railway comes as a result of its advantages, namely:

- Flexibility to adapt to the demand
- Dynamic comfort of passengers
- Relatively high commercial speed
- Great passenger transport capacity
- Reliable services
- Frequent services
- Increased transport safety
- Low burden to the environment

8.3.2 Disadvantages

The main disadvantages of the suburban railway are

- The disturbance caused during its construction
- The high construction cost of the infrastructure (€10–30 M per track-km, data 2014), and that it is reduced significantly when the existing infrastructure is used

8.4 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

The suburban railway can operate by using either existing or new infrastructure.

In the case where suburban services are provided on the existing infrastructure, the following issues must be addressed (Pyrgidis, 2008):

1. Can the length of the connection justify suburban services?
2. Are there any attractive advertisements and interest generating items/signages placed at intervals along the rail route?
3. Are the existing passenger transport volume and the potentially developed one, significant? Is there seasonal fluctuation in the transport volume? Is there any demand for transport between intermediate stops?
4. Does the railway infrastructure allow the development of the desired speeds and, subsequently, the desired travel times?
5. Is there sufficient available rolling stock suitable for suburban services (frequent services, 'dynamic comfort' of passengers, etc.)?
6. Do the stations provide a high level of service to the passengers? Is there the possibility of transfer and, more generally, of integration with other means of transport (park and ride service facilities, buses, etc.)? Are the railway stations located in close proximity to the city centres that need to be served? Are the stations and the stops safe?
7. Does the capacity of the line allow the operation of additional trains and, specifically, those that are required in order to achieve frequent services? Do trains of any other category run on the line (freight, interurban, high-speed trains), and if so, are the timetables well harmonised?
8. Is the line electrified, in order for the desired traction characteristics to be assured and, most importantly, in order to avoid any environmental consequences at the transit area as a result of the increased number of trains?
9. Is the line equipped with electric signalling system? Is there an Automatic Train Protection System additionally installed?
10. Are there many level crossings (if any)? Are they equipped with automatic barriers? Is there any fencing along the line?
11. Is it possible to ensure that there is adequate space for the construction and the effective operation of areas of maintenance, repair and parking of rolling stock for the specific network of suburban service?
12. Does the operation of trains create problems of noise pollution and vibrations that can be successfully addressed?
13. Can the issues 4–12 be dealt with using relatively low-cost interventions?
14. And importantly, are the users of the other competing means of transport in favour of the creation of a suburban railway in that area?

In the case of a new infrastructure, the issues that must be examined are 1, 2, 3, 11, 12, 13 and 14, whereas all the rest must conform to the specifications to which the construction is tendered.

8.5 APPLICABILITY VERIFICATION OF THE SYSTEM

As in the case of the tramway (see Section 4.8) this process imposes for each of the alternative alignments, the individual verifications of applicability that are given in the logical diagram of Figure 8.3, and which must be simultaneously satisfied (Pyrgidis, 2008).

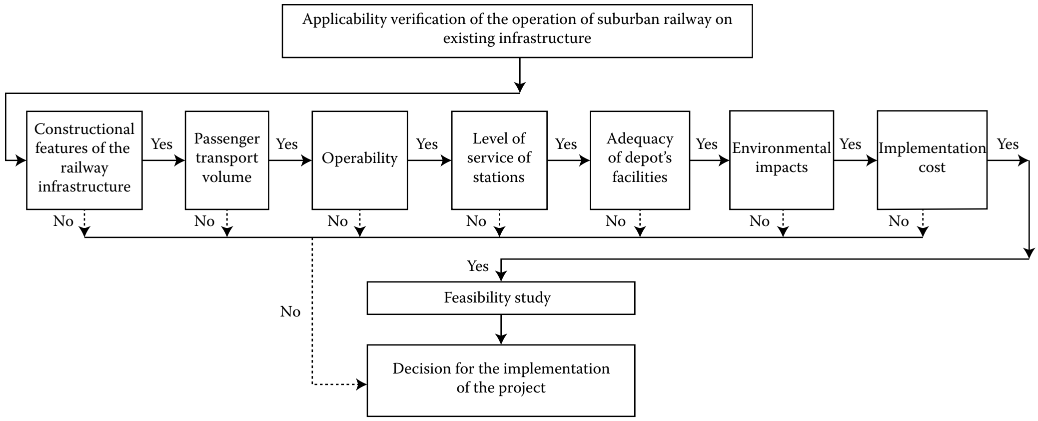

Figure 8.3 Operation of suburban railway on existing infrastructure – Logical diagram of project implementation acceptance.

8.5.1 Operation of suburban trains on existing infrastructure

In the case where the operation is scheduled to take place on existing railway infrastructure, all the issues cited in Section 8.4 must be satisfied.

8.5.1.1 Constructional features of the railway infrastructure

This applicability verification examines the issues 1, 2, 4, 8, 9 and 10 of Section 8.4.

It includes the following partial verifications:

1. Verification of connection length
2. Verification of the number of tracks
3. Verification of the average distance between intermediate stops
4. Verification of the track geometry alignment design (alignment radii, longitudinal gradients)
5. Verification of the various components of the track superstructure
6. Verification of the signalling system
7. Verification of the traction system
8. Verification of the fencing
9. Verification of the average running speed

The following steps are applied to verify the average running speed (V_{ar}):

- Definition of the desired travel time t (target time) for the connection length S (taking into account the travel time and particularly the travel time that is required by the competitive means of transportation)
- Definition of the number of intermediate stops n_s and the halt time t_s at each stop
- Calculation of the average running speed of trains V_{ar} that is required in order to achieve the target time, by using Equation 8.1

$$t = (S/V_{ar}) + n_s \cdot t_s \tag{8.1}$$

- Recording of the permitted running speed of trains along the existing line V_{maxtr} and calculation of the average permitted speed V_{amaxtr} for the total connection length
- Comparison between V_{ar} and V_{amaxtr}

The verification is considered positive when $V_{ar} \leq V_{amaxtr}$. Otherwise, necessary interventions must be made to the existing railway superstructure so as to secure the V_{ar} (e.g., by reducing the number of intermediate stops, by increasing the quality level of maintenance of the track, by improving the geometry of the track alignment).

8.5.1.2 Passenger transport volume

This verification examines the issues 3 and 14 in Section 8.4, and it also provides data that helps in conducting other verifications.

In order to conduct this verification, the following are required:

- Daily ridership measurements including all transportation modes along the route. These measurements should cover both peak and off-peak periods.
- A questionnaire survey addressing the users of the existing transportation modes along the connection. This survey must use well-structured questionnaires.
    - Recording of information on the passenger flow characteristics and, particularly, origin–destination, trip purpose and trip frequency.
    - Recording of users' opinion concerning the development of a suburban railway system and their intention to use this system if it shall operate with the foreseen design characteristics.

A combination of the two above-mentioned individual verifications provides an initial estimate of the number of transported passengers that are expected to use the suburban railway service (daily ridership, passenger/h/direction), as well as an indication of whether there will be seasonal fluctuation of the transport volume, and whether there will be adequate transport demand between the intermediate stops of the connection.

At this point, it should be clarified that the measurements and the questionnaire surveys conducted in this phase do not substitute those to be conducted in the phase of the comprehensive feasibility study.

In such case, by suitable processing of the data acquired during the measurements and the questionnaire surveys, and by making some simplified assumptions, the potential transport volume of the suburban railway service (total number of passengers P_d carried per hour and daily) can be estimated.

Concerning the ridership, values in the range of 5,000 passengers/h/direction are considered acceptable for the suburban railway, while in the case of a regional railway, the respective values are lower (Batisse, 1999).

8.5.1.3 System operability

This verification concerns the issues 5 and 7 in Section 8.4. It includes four individual verifications.

8.5.1.3.1 Verification of the system's passenger transport capacity

During this verification:

a. Knowing the desired travel time (one-way trip)
b. Having selected

- The type and formation of the trains
- The passenger capacity of the trains (this estimation depends on the number and the transport capacity of the vehicles in relation with the acceptable passenger density)
- The waiting time of the trains at the two terminal stations
- The network operation hours during the day (total of 24 h)

c. And having considered various scenarios related to the headway of trains at peak and off-peak hours
   - The passenger transport capacity of the system P'_d is calculated (passenger/h/direction and total per day)

In the case where the resulting total daily passenger transport capacity P'_d is smaller than the potential transport volume of the suburban railway service P_d, one can then adjust one of the parameters (b) or (c) in an effort to satisfy the anticipated demand. The choice of double-deck trains is always a solution. Yet, in this case, a verification of the structure gauge must also be conducted.

8.5.1.3.2 Track capacity verification

The track capacity must allow the operation of additional trains. Its calculation can be performed by using one of the existing capacity calculation methods (Kontaxi and Ricci, 2009).

8.5.1.3.3 Verification of required rolling stock

The theoretically required number of trains is calculated by dividing the two-way route travel time (round-trip plus waiting time at the two terminal stations) by the headway of trains at peak hours, and rounding it up to the nearest integral values. The finally required rolling stock should allow for reserves for maintenance and reserves in the event of a breakdown (Baumgartner, 2001).

8.5.1.3.4 Verification of availability and adequacy of rolling stock

This partial verification comprises the following procedures:

- Definition of the basic specifications to be met by the rolling stock in order for the above verifications to have positive results (rolling stock design speed, traction characteristics, train type, train formation and passenger transport capacity, compatibility between the vehicle's floor height and the platform's height, compatibility between the vehicle's dynamic gauge (width) and its gap from the platforms).
- Inventory of the available rolling stock which satisfies the aforementioned standards with such features. At this point, it should be considered that, in the target year, all railway vehicles in service will have an age of 15 years or less (30 years usually being the useful lifespan of a railway vehicle).

The necessary fleet is estimated for the target year by subtracting the available rolling stock in the target year from the finally required rolling stock.

8.5.1.4 The station service level

This verification examines issue 6 of Section 8.4. It includes the following individual verifications:

1. Verification of connectivity with other transport systems and, in general, of park and ride service facilities
2. Verification of location of the stations/stops
3. Verification of the security and of the services that are provided in the stations/stops

8.5.1.5 Availability of the depot facilities

This verification deals with issue 11 of Section 8.4.

When a relevant infrastructure exists, this verification can also examine whether those can meet the needs of the new railway service.

8.5.1.6 Environmental impacts

This verification examines issue 12 of Section 8.4.

When trains operate on existing infrastructure, the main possible impact is the increase of noise pollution and vibrations due to the increased train traffic. It is estimated that these problems can often be addressed with financially accepted solutions, unless the length of the line where interventions are needed is considerably extensive (see Chapter 19).

8.5.1.7 Implementation cost

This verification examines issue 13 of Section 8.4.

When trains operate on existing infrastructure, limits cannot be placed with regard to the acceptable implementation cost. In general, in such cases, the cost is significantly lower than the respective cost of contemplating the construction of a new infrastructure.

8.5.2 Operation of suburban trains on new infrastructure

As already mentioned in Section 8.4, when the construction of new infrastructure is contemplated, the issues to be examined are issues 1, 2, 3, 11, 12, 13 and 14, while all the others must be fulfilled by the project-tendering specifications.

In any case, the selected track design speed V_d must ensure the desired travel time (issue 4). In this context, the commonly applied practice is to calculate the average running speed V_{ar} by applying the mathematical Equation 8.1.

The track design speed V_d is calculated as a percentage of V_{ar} (e.g., 1.25% of its value) and, particularly:

$$V_d = V_{ar}/0.8 \tag{8.2}$$

The value of V_d is calculated by rounding up the result of the mathematical Equation 8.2 and by selecting values of V_d equal to 120, 140, 150, 160 km/h.

8.5.2.1 Constructional features of the railway infrastructure

This verification deals with issues 1 and 2 of Section 8.4.

It includes two individual verifications:

- Verification of connection length
- Verification of average distance between intermediate stops

8.5.2.2 Passenger transport volume

As in the case of the existing infrastructure (Section 8.5.1.2).

8.5.2.3 Location, construction and operation of the depot facilities

This verification examines issue 11 of Section 8.4.

It is a prerequisite that the functional connection of the main traffic line with an area should be where the following activities can be performed rationally:

- Maintenance, repair and washing of rolling stock
- Parking of rolling stock

The above sites are mainly sought out at the end of the lines and (or) at places where empty runs are minimised. Their dimensioning depends on the size of the fleet of the rolling stock, the length of the trains and, mainly, the future network extension plans.

8.5.2.4 Environmental impacts

This assessment examines issue 12 of Section 8.4.

When the construction of a new infrastructure is contemplated, problems may arise during both the construction and the system operation phases. The most common impacts relate to noise and visual annoyance (see Chapter 19).

The object of environmental applicability verification is to examine to which extent the environmental engineer considers such consequences minor in relation to the functionality of the network or the capability of generating a radical reorganisation of the project's design.

8.5.2.5 Implementation cost

This verification deals with issue 13 of Section 8.4.

The implementation cost includes the cost of construction of the infrastructure and the cost of purchase of the rolling stock. The verification requires a first approximate evaluation of the cost of the project and the comparison of the investment amount with an average value that is considered to characterise similar projects. An amount of €15 M per km of track can be adopted as such a value. This price comprises the track, the signalling/electrification/telecommunication systems, the station's facilities, and the electromechanical equipment (that is to say, without taking the cost of rolling stock purchase into account).

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Chapter 9

Rack railway

9.1 DEFINITION AND DESCRIPTION OF THE SYSTEM

The operational limits and the economic exploitation of a rail transport system are directly dependent on the track's geometrical design. More specifically, the greatest problems are caused by the large longitudinal gradients. In these cases, the movement is a function of the available traction power, the weight of the rolling stock and the wheel–rail adhesion.

The coefficient of friction between steel and steel cannot exceed a certain value. Therefore, the longitudinal gradient of the railway track which allows for the economic exploitation of the network is predefined (see Table 1.3).

In the case of gradients that are greater than 50‰–70‰, an additional force is required on inclines in order to overcome the force of gravity. This force is added to the movement resistance. The required braking force, which depends on the gradient and, consequently, the reliability of the braking system also plays an important role.

To ensure the required additional traction and braking force, two techniques are used:

- The cog railway
- The traction by cables (cable-propelled systems, see Chapter 10)

The track superstructure of a rack railway consists of two conventional rails plus a toothed rack rail in-between (Figures 9.1 and 9.2). The wheelsets of the powered vehicles are equipped with one or more cog driving wheels that are arranged either horizontally or vertically (Figure 9.3). The vehicles (powered and trailers) are usually equipped with cog braking wheels, which along with the driving wheels mesh with the rack rail in-between the two main rails, and thus ensure the necessary supplementary traction and braking efforts.

This transport system is mostly used in mountainous areas and tourist resorts, when the track alignment includes longitudinal gradients greater than 50‰–70‰.

9.2 CLASSIFICATION OF RACK RAILWAY SYSTEMS

9.2.1 Type of cog rail

During the development of rack railways, there were three cog rail systems that were mainly used. The most commonly used cog system nowadays is the Abt system. The other two well-known systems are the Riggenbach system and the Strub system.

The Abt system was invented by Roman Abt in 1882, a Swiss locomotive engineer (Wikipedia, 2015b). This system limits the discontinuities that arise when the traction effort is applied (Figures 9.4 and 9.5).

Figure 9.1 Track superstructure of a rack railway, Chamonix, France. (Photo: A. Klonos.)

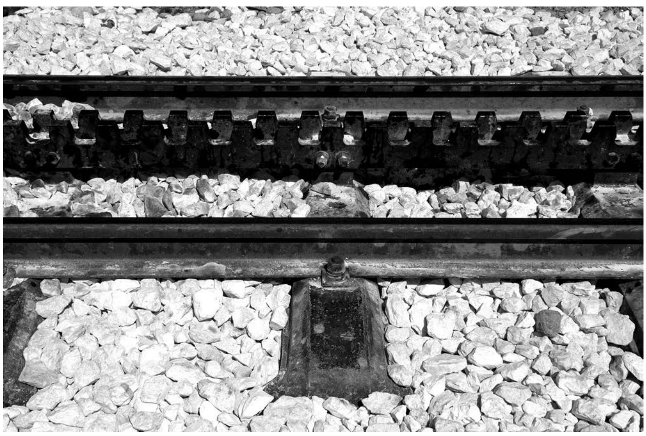

Figure 9.2 Track superstructure of a rack railway, OSE, Kalavryta, Greece. (Photo: A. Klonos.)

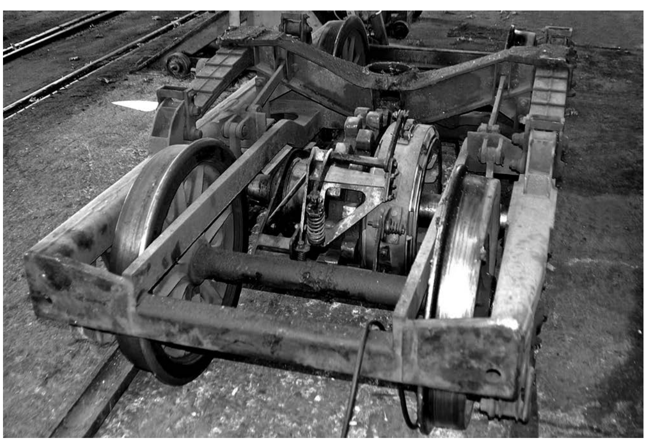

Figure 9.3 Driving axle of a powered rack railway vehicle, OSE, Greece. (Photo: A. Klonos.)

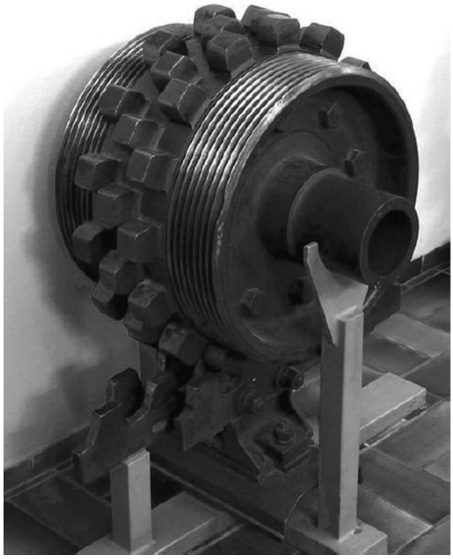

Figure 9.4 Cog wheel used on Abt system. (Adapted from Softeis in Deutschen Museum, 2004, Online image available at: en.wikipedia.org/wiki/Rack_railway (accessed 8 August 2015).)

The Riggenbach system was invented by Niklaus Riggenbach in 1863. The system is more complex and expensive to build than the other systems (Wikipedia, 2015b). However, it can sustain greater forces while effectively keeping the wear at a minimum level.

Figure 9.6 shows a cog wheel using the Riggenbach system, while Figure 9.7 illustrates a track equipped with the Riggenbach system.

The Strub system was invented by Emil Strub in 1896. It is the simplest of all rack systems in use and has become increasingly popular (Wikipedia, 2015b).

Figure 9.8 shows a cog wheel using the Strub system, while Figure 9.9 illustrates a track equipped by the Strub system.

Among other less used rack systems are Locher (also known as Punch system) (Figure 9.10) (Wikipedia, 2015a), Lamella (also known as the Von Roll system; Figure 9.11), Riggenbach Klose, Marsh and Morgan (Figure 9.12).

Figure 9.5 The Abt cog system, OSE, Kalavryta, Greece. (Photo: A. Klonos.)

Figure 9.6 Cog wheel used on Riggenbach system, Switzerland. (Photo: A. Klonos.)

Figure 9.7 The Riggenbach cog system, Arth Goldau, Switzerland. (Photo: A. Klonos.)

Figure 9.8 Cog wheel used on the Strub system, Switzerland. (Photo: A. Klonos.)

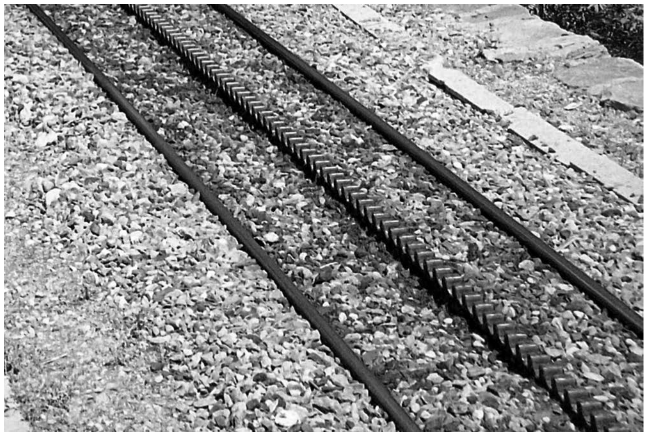

Figure 9.9 The Strub cog system, Chamonix, France. (Photo: A. Klonos.)

Figure 9.10 Cog wheel used on Locher system, Switzerland. (Photo: A. Klonos.)

Figure 9.11 The Lamella cog system, Capolago, Switzerland. (Photo: A. Klonos.)

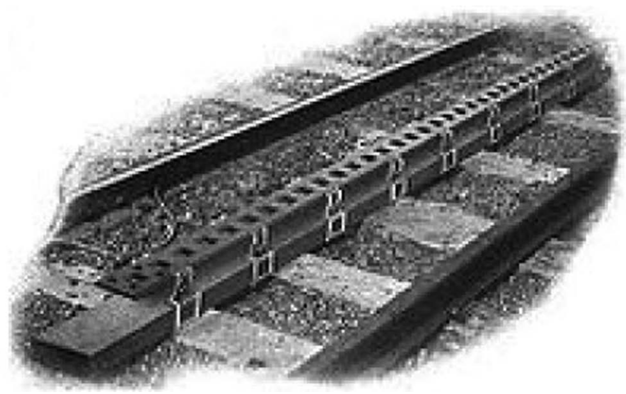

Figure 9.12 The Morgan cog system. (Adapted from Goodman Manufacturing Company, 1919, *Goodman Mining Handbook for Coal and Metal Mine Operators, Managers, etc.* Online image available at: en.wikipedia.org/wiki/Rack_railway (accessed 8 August 2015).)

The Locher 'herringbone' system is implemented with a pair of opposed horizontal pinions engaging teeth cut into the sides of the rack, so that it does not depend on gravity for the engagement of the teeth, and can be used on extreme longitudinal gradients. The sole example is the Pilatus cog railway, Switzerland, 1885, with a maximum gradient of 480‰. A similar system is that of the Krasnoyarsk ship lifting line in Russia. Finally in Panama a special system that resembles the Riggenbach system is used (Treidellokstrecken).

Overall, out of a total of 65 rack railways (data 2014), the most frequently used system is the Abt system (about 40%), followed by the Riggenbach system (23%) and the Strub system (17%).

The Lamella system is found in three lines, while the Locher and Marsh systems are found in one line (Pilatus Bahn and Mount Washington Cog Railway, respectively). A combination of the Strub and Lamella systems is applied in two lines. A combination of Riggenbach and Lamella systems is applied in four lines. Finally, a combination of the Riggenbach, Lamella and Strub systems is applied in one line (St. Gallen-Gas-Appenzell, Switzerland) (Jehan, 2003).

All the above data and the data recorded and analysed in the following related to the year 2014. The raw data were obtained both per country and per line, from various available sources, and were cross-checked (Syofiardi Bachyul, 2009; Wikipedia, 2015a–c; Bruse's Funiculars Net, no date). Afterwards they were further manipulated for the needs of this chapter.

9.2.2 Type of adhesion along the line

The rail systems using the rack/cog rail are split into two categories based on the type of adhesion:

- Purely toothed (racked) tracks
- Mixed adhesion operation (dual rack and adhesion system)

The first category consists of all the tracks in which the rolling of vehicles throughout the entire line is ensured by artificial adhesion (rack rod/rail). These lines typically include comparatively very high longitudinal gradients, serve much shorter distances, and usually connect mountainous tourist resorts with the flat areas below them.

The second category comprises the lines that include track sections of very steep gradients, in which the traction of vehicles is ensured additionally by the use of a rack rail, but also sections where rolling is ensured through conventional adhesion only (Machefer Tassin, 1971; Dunn, 1980; Avenas, 1984). This category covers a wider range of lines (touristic, urban, etc.).

For these networks, special systems have been designed for the rolling stock, which allow an easy transition from two to three rails while the train is in motion as inclines increase (Carter and Burgess, Inc., 2006).

From a total of 65 rack railway systems worldwide, 35 (2014 data) are dual rack and adhesion systems while 30 are purely racked.

9.3 EVOLUTION OF THE SYSTEM AND APPLICATION EXAMPLES

The very first cog railway was opened in 1869 on Mount Washington in New Hampshire, USA, and still operates with its original steam locomotives. The idea to build a cog railway up to the top of the mountain was attributed to Sylvester Marsh in 1857.

Rigi-Bahnen (Rigi railways) in Switzerland opened in 1873 as the first cog railway to operate in Europe (Jungfrautours, no date).

By the end of the nineteenth century, rack railways had spread rapidly all over Europe and America. The main reason for this rapid expansion was the fact that at that time conventional steam locomotives were not able to pull heavy loads on gradients higher than 45‰. This was to change in the next century with the development of advanced electric locomotives, which could cope with inclinations of up to 100‰ without the need of rack wheels. Thus, the majority of rack railway lines were consequently replaced, closed down, or continued operation for touristic reasons; mostly those of relatively low longitudinal gradients.

During the last 25 years, only three new rack railway lines have been built and put into operation:

- The Quincy and Torch Lake Cog Railway (Hancock – Quincy Mine's), in the USA, 701 m long (1997)
- The Skitube Alpine Railway (Bullocks Flat – Perisher Valley – Blue Cow) in Australia, 8,500 m long (1988)
- The Panoramique des Domes line in France, 5,200 m long, (May 2012)

It should be noted that some lines that had seized operation were refurbished and commenced operation again (for example the Montserrat line in Barcelona was closed in 1957 and it recommenced operation in 2003) or are expected to be fully operational in the near future (for example the Da La line in Vietnam).

Table 9.1 presents the number of cog railway systems in service in the world, per continent and per country. The vast majority of these systems are located in Switzerland.

9.4 CONSTRUCTIONAL AND OPERATIONAL FEATURES OF RACK RAILWAY SYSTEMS

9.4.1 Track alignment

The cog rail technique can be applied to all track gauges that are characterised as 'small' track gauges. It performs better in tracks of metric and narrow gauges (1.00, 0.80,

Table 9.1 Number of cog railway systems in service per continent and per country (2014 data)

Continent	*Country*	*Number of cog railway systems with purely rack tracks*	*Number of dual rack and adhesion systems*	*Total*
Europe	Austria	2	1	3
	Czech Republic	0	1	1
	France	2	3	5
	Germany	2	2	4
	Greece	0	1	1
	Hungary	1	0	1
	Italy	3	0	3
	Slovakia	1	1	2
	Spain	0	2	2
	Switzerland[a]	12	18	30
	Russia	1	0	1
	United Kingdom	1	0	1
America	Brazil	1	1	2
	Panamas	0	1	1
	USA	3	0	3
Asia	India	0	1	1
	Indonesia	0	1	1
	Japan	0	1	1
Oceania	Australia	1	1	2
Total		30	35	65

[a] The Monte Generoso line is under refurbishment and is expected to commence operation in 2016.

0.75 m) because the reduced size and weight of the vehicles require smaller traction and braking efforts.

The most commonly used track gauge in the field of cog railway systems is 1,000 mm (52.3%), followed by 1,435 mm gauge (17%) and, thirdly, 800 mm gauge (12.3%). The Krasnoyarsk ship lifting line in Russia has a 9,000 mm gauge.

Rack railway lines usually involve a single track, and very steep longitudinal gradients, which determine the maximum speed in ascent and descent. The longitudinal gradient can range from 50‰ to as much as 480‰, while usual values lie between 200‰ and 250‰. The steepest cog railway in the world is Pilatusbahn in Switzerland, with maximum gradient 480‰. In station areas, gradients of 20‰–70‰ are allowed.

The total length of the 30 purely rack lines in service is approximately equal to 177,000 km. The length of these lines usually ranges from 4,500 to 6,000 m. Wengernalpbahn in Switzerland is the world's longest cog railway with a continuous toothed line of 19,090 m length.

Dual rack and adhesion tracks usually exhibit much longer lengths.

The horizontal alignment of a cog railway can include curve radii not lower than $R_c = 85–90$ m. The cant is calculated in the same way as in the case of secondary, local, or regional railway lines.

9.4.2 Track superstructure

Rack railway tracks are usually placed at grade. However, there are also some examples of underground tracks (e.g., the Skitube Alpine line in Australia, with a length of 8.5 km

Figure 9.13 The Strub cog system switch, Wengernalpbahn, Switzerland. (Photo: A. Panagiotopoulos.)

that features two tunnels that are 3.3 km long [Bilson Tunnel] and 2.6 km long [Blue Cow Tunnel], respectively).

Given that most tracks are integrated in difficult mountainous terrain, a specific track bed design is needed. The minimum depth of the ballast – measured from the bottom to the underside of the sleepers – is 20 cm. The rails are common wide foot rails, and need to have higher mechanical resistance than conventional rails. The distance between sleepers follows the same principles as in the case of conventional adhesion tracks. The flange clearance 'σ' must not exceed 5 mm.

The use of toothed tracks necessitates a more sophisticated implementation of switches and crossings (Figures 9.13 through 9.15).

The construction cost of a purely toothed cog rail single track infrastructure normally varies between €10 M and €15 M/km (2014 data) (Carter and Burgess, Inc., 2006; http://www.revolvy.com/main/index.php?s=Panoramique).

Figure 9.14 The Abt cog system switch. (Adapted from Hapestof, 2009, Online image available at: http://cs.wikipedia.org/wiki/Ozubnicov%C3%A1_dr%C3%A1ha (accessed 8 August 2015).)

Figure 9.15 The Riggenbach cog system switch. (Adapted from Hapestof, 2009, Online image available at: http://cs.wikipedia.org/wiki/Ozubnicov%C3%A1_dr%C3%A1ha (accessed 8 August 2015).)

9.4.3 Rolling stock

Most purely toothed cog railway trains comprise one or two vehicles, with a total transport capacity of 100–200 passengers (seated and standing).

When designing rolling stock intended to operate on steep longitudinal gradients, particular effort is put into minimising the weight of the vehicles.

The power vehicles are equipped with toothed wheels to enforce traction and braking. The trailer vehicles have at least one toothed wheel per axle to ensure proper braking and to avoid slipping when moving downhill. Moreover, the power vehicles are placed, for safety reasons, either in the rear when ascending (pushing the other vehicles), or in the front when descending (pulling the other vehicles).

The safety operation of the system necessitates a reliable braking system, capable of functioning properly under any climatic or operational conditions. Therefore, rolling stock intended for high longitudinal gradients is equipped with

- A 'main' brake, which enforces constant speed when descending.
- Two independent brakes that apply on the driving toothed wheels or on the braking wheels, and which can effectively immobilise the vehicle in the most adverse section of the track.
- A braking system which is automatically activated once the train speed exceeds the allowed speed limit. This system is indispensable in lines with gradients $i > 125‰$.
- A system that allows the automatic braking of the powered vehicles, while traction is still applied.
- A supplementary safety brake which, in case of steep acclivities, prevents the gravity-triggered backward movement of the vehicles.

In certain cases of very steep gradients, the passenger seat level – for obvious reasons – is inclined with regard to the track level (Figure 9.16).

The touristic nature of most cog railways requires increased visibility from inside the vehicles over the picturesque landscapes and highlands that they cross. This need for panoramic views translates into much larger window surfaces than in conventional vehicles.

Figure 9.16 Seat arrangement in cog railway vehicles in case of very steep gradients, Pilatusbahn, Switzerland. (Photo: A. Klonos.)

All modes of traction have been used in rack railways: steam, diesel and electricity.

In case of electric traction, the overhead catenary would require special attention during inclement weather which could increase the cost of this alternative. Diesel units do not feature this problem, but may have additional problems, such as additional noise and air pollution (Carter and Burgess, Inc., 2006).

However, electric traction features a major advantage in comparison with diesel traction. In diesel traction, the braking equipment is rapidly discarded because of the sheer amount of consumed energy in order to retain constant speed when descending. In electric traction though, this problem is eliminated, since it is possible to recuperate part of the energy that is wasted during braking.

Historically, the first electric cog railway was constructed in Salève (1891–1893) by the engineer Thurry. The oldest steam cog railway in Europe is Achenseebahn, in Austria (opened in 1889).

Out of a total of 65 rack railway systems (2014 data), the majority are exclusively electrically powered (69%). Out of these systems, four use three-phase current. Finally, five systems are exclusively steam loco-hauled.

Biodiesel locomotives operate in Mount Washington and Manitou Pike's Peak Cog Railway (USA).

The cost of cog railway rolling stock is very high (€3–5 M per train) (2014 data).

9.4.4 Operation

Cog railway systems can be divided into two categories from the operation point of view. The first is related to providing year-round operation for daily passenger transport. The second category refers mainly to tourist traffic, where in some cases, the cog railway itself is the attraction.

According to 2014 data, 90% of cog rail systems operate throughout the whole year, and the remaining operate seasonally (usually from March to October).

Purely toothed cog railways usually operate at speeds of $V = 15–25$ km/h (maximum speed can go up to $V_{max} = 40$ km/h) when ascending. When moving downhill however, safety restrictions impose lower speeds. The commercial speed of a cog railway system varies from $V_c = 7.5$ to 20 km/h.

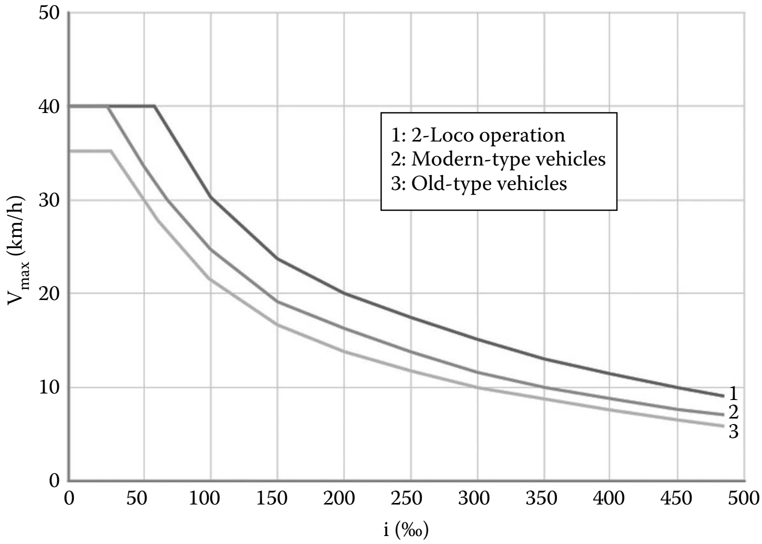

Figure 9.17 Maximum allowed speed V_{max} for cog railway in descent, as a function of longitudinal inclination i of the track (‰). (Adapted from Loosli, H. 1984, *Le chemin de fer à crémaillère – ses particularités et domaines d'application*, Revue Technique Sulzer, No 2, pp. 17–20.)

In dual rack and adhesion systems, trains run at V = 20–30 km/h on the toothed sections (speeds up to 40 km/h have been recorded). Maximum speeds of V_{max} = 75 km/h have been recorded on the conventional track sections.

Figure 9.17 illustrates the allowed descending speed as a function of the longitudinal gradient (‰). Curve 2 corresponds to modern-type vehicles, curve 1 with old-type vehicles, and curve 3 relates to 2-loco traction (dual-power vehicles) (Loosli, 1984).

The frequency of the service depends mainly on the demand and on the kind of services offered.

Rack railway provides the possibility of intermediate stops; the transport system capacity is a function of

- The connection length S
- The commercial speed V_c
- The vehicle transport capacity C_v
- The dwell time t_{ts} at the two terminal stations

Considering that S = 6,000 m, V_c = 15 km/h, C_v = 200 passengers and t_{ts} = 6 min in each terminal, the system's transport capacity is calculated at 400 passengers/h/direction.

Table 9.2 provides the main characteristics of some purely racked lines indicatively.

9.5 ADVANTAGES AND DISADVANTAGES OF RACK RAILWAY SYSTEMS

Advantages:

- Cog railway operation is not affected by climatic conditions and/or by the weather, and it is also considerably environmentally friendly.

Table 9.2 Main characteristics of purely racked lines (indicatively)

Line/country	*Cog system*	*Length(m)*	*Traction system*	*Max gradient (%)*	*Track gauge (mm)*
Petit train de la Rhune (France)	Strub	4,200	Electric (3-phase)	25–27	1,000
Stuttgart Rack Railway (Germany)	Riggenbach	2,200	Electric	17.5	1,000
Schafberg Railway (Austria)	Abt	5,850	Steam + diesel	26	1,000
Jungfraubahn (Switzerland) (the rack railway that operates at the greatest height in Europe)	Strub	9,300	Electric (3-phase)	25	1,000
Pilatus Bahn (Switzerland) (the steepest rack railway in the world)	Locher	4,800	Electric	48	800
Wengernalpbahn (Switzerland)(the longest continuous rack railway in the world)	Riggenbach + Lamella	19,090	Electric	25	800
Snowdon Mountain Railway (UK)	Abt	7,600	Steam + diesel	18	800
Fogaskerekű Vasút (Hungary)	Strub (specific profile)	3,700	Electric	11	1,435
Štrbské Pleso-Štrba (Slovakia)	Lamella	4,757	Electric	12,7	1,000
Principe-Granarolo (Italy)	Riggenbach	1,136	Electric	21.4	1,200
Corcovado Train (Brazil)	Riggenbach	3,800	Electric (3-phase)	30	1,000
Mount Washington Cog Railway (USA)	Marsh	4,800	Steam + biodiesel	37.4	1,422
Skitube Alpine Railway (underground 5.9 km) (Australia)	Strub + Lamella	8,500	Electric	12.5	1,435

- It is suitable for lines with varying longitudinal gradients, as it offers the possibility of operating both in conventional and artificial adhesion. This significantly reduces the amount of earthworks required, and allows for the design of a line that offers a good cost/utility ratio.
- It allows for connection with a pre-existent network without excluding the possibility of future extension.

Disadvantages:

- The cost of rolling stock is high due to the specialised equipment.
- The running speeds and the transport capacity of cog railway are significantly smaller when compared with conventional railways.
- It requires specialised staff for operation and maintenance.
- The braking system is complicated.
- Switches and crossings display technical difficulties in realisation.

9.6 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

In general, purely toothed cog railway is preferred when considering distances of S = 4–20 km, with constant longitudinal gradients of i = 50‰–250‰ (max 480‰), and relatively high transport demand. It is mostly used for passenger transport, and in very few cases it is also used for freight. Its contribution is mainly directed toward leisure activities (tourism, excursions).

Alternatively, the problem of steep longitudinal gradients can be solved by cutting through the cliff mass (tunnel), or by helical or spiral track alignment design (e.g., St. Gotthard Pass in Switzerland), or by use of successive course-reversing stubs (switchback-design type, ‘z’), as in the case of the rail pass through the Andes in Peru.

Dual rack and adhesion systems are preferred for connections of S = 10–50 km, which include sections of high longitudinal gradients that normally limit conventional adhesion. In this case, mixed adhesion operation very often provides a more efficient solution economy-wise, than changing the track alignment or introducing more powerful locomotives.

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Chapter 10

Cable railway systems for steep gradients

10.1 DEFINITION AND DESCRIPTION OF THE SYSTEM

For movement on longitudinal gradients that are greater than gradients on which a conventional railway may climb, apart from the rack rail technique, cable hauling can also be used.

On the basis of the technique that is applied for vehicles' traction, cable railway systems for steep gradients are classified into three categories:

- Funicular (non-detachable cable-propelled vehicles for steep gradients)
- Cable railway (detachable cable-propelled vehicles for steep gradients)
- Inclined elevator

The funicular (or inclined plane, or inclined railway or cliff railway) (Figure 1.37) operates using two vehicles which move on rails with the aid of a cable; one of the vehicles is ascending while the other one is descending. The cable rolls over pulleys mounted on the track (Figures 10.1 and 10.2). The smooth movement of the cable and of the whole system is ensured by means of an electric motor, placed at the highest point of the network. The vehicles are permanently connected to both ends of the cable and they start and stop simultaneously. The ascending vehicle uses the gravitational force of the descending one (counterbalance system). This system connects distances $S < 5$ km, which have continuous gradients that usually vary between 300‰ and 500‰ (maximum recorded gradient: $i_{max} = 1{,}200$‰).

The cable railway also uses vehicles which run on conventional rails with the aid of a cable which is moving constantly and at a constant speed. The difference between the two systems is that for the cable railway the vehicles can be detached from the traction cable as they are not permanently connected to it. The vehicles can stop independently by detaching from the cable, and they may start again by reattaching to the cable. This process can be done automatically and manually (San Francisco cable-propelled system, USA).

The inclined elevator (or inclined lift or inclinator) is a variant of the funicular. It operates using a single vehicle. The vehicle is either winched up at the station at the top of the inclined segment where the cable is rolled around a winch drum, or the weight of the vehicle is balanced by a counterweight, hence the system operates as a funicular. It usually connects distances $S < 1.5$ km (maximum recorded value $S_{max} = 1.6$ km) that have large continuous gradients (typically 500‰–700‰) which may reach significantly high values (systems Eiffel Tower, Paris, France: $i_{max} = 4{,}761$‰; Olympic Stadium, Montreal, Canada: $i_{max} = 1{,}880$‰; Katoomba Scenic, Australia: $i_{max} = 1{,}288$‰).

The oldest funicular system for large longitudinal gradients in the world is the inclined elevator of Reisszug (Wikipedia, 2015d). This is a private single track with a length of $S = 190$ m and a gradient of $i_{max} = 670$‰ which serves the transport of people (three passengers) and goods (2,500 kg) to the Hohensalzburg Castle in Salzburg, Austria. It was first documented

Figure 10.1 Cable-guiding pulleys, Barcelona, Spain.

Figure 10.2 Cable rolling through pulleys, Saint Moritz, Switzerland.

in 1515. The track originally used wooden rails and a hemp hauling rope, while the mechanism itself was being operated by horses or people. Today steel rails, steel cables and an electric motor have replaced the old technology, but the track still follows the same route.

10.2 THE FUNICULAR

10.2.1 Evolution of funiculars and application examples

Modern funicular railways operating in urban areas date from the 1860s. The first line of the funiculars of Lyon opened in 1862, followed by other lines in 1878, 1891 and 1900.

The Giessbachbahn railway constructed in 1879 in Switzerland was Europe's first funicular railway used for transporting passengers.

A total of 249 funicular systems are recorded in the world. These systems are either in operation or under maintenance, and are expected to reopen immediately (Wikipedia, 2015a,b; Bruce's Funiculars Net, no date). Table 10.1 presents the number of 'funiculars' per continent and per country. The majority of 'funiculars' in operation are in Switzerland (49), followed by Italy (26) and Japan (20).

All the above data and the data recorded and analysed in the following relate to the year 2014. The raw data were obtained both per country and per line, from various available sources, and cross-checked. Thereafter, they were further manipulated for the needs of this chapter.

10.2.2 Constructional and operational features of funiculars

10.2.2.1 Infrastructure

The operation of all funicular railway systems for steep gradients is performed using the Shuttle 'principle' (see Section 7.2.2.1).

Regarding the track superstructure configuration, it is either single-lane shuttle with bypass type or dual-lane shuttle type.

In the first case, the system operates using two vehicles which intersect while moving between terminals, along a single track, using a pull cable. The passing loop is located halfway.

In the second case, the system operates using two vehicles which move between terminals, along a double track without passing loop, using a pull cable.

In both cases, the counter-balance system is used, and the intermediate stations (0–3) are located at symmetrical distances.

With regard to the number of rails used to form the track superstructure, funiculars can be classified into three categories.

Two-rail superstructure configuration with passing loop (Figures 10.3c, 10.4, and 10.5): This type of superstructure does not require switches and crossings. The vehicles feature specially arranged wheels (see Figure 10.3c). More specifically, only the wheels of one side of the wheelset feature a flange which is in fact double. The intersecting vehicles have their wheels with a flange at opposite sides, and this allows them to follow different tracks. This configuration significantly reduces costs.

Three-rail superstructure configuration with passing loop (Figures 10.3b and 10.6): In superstructure layouts using three rails, the middle rail is shared by both cars. The superstructure is wider than the two-rail layout, but the passing loop section is simpler to build.

Four-rail superstructure configuration (Figures 10.3a and 10.7): In some four-rail funiculars (Figure 10.7), the upper and lower sections are interlaced while having a single platform at each station. Table 10.2 presents the basic constructional and operational characteristics of *funiculars*. Funiculars operate on tracks with various gauge values. The most common gauge value is 2e = 1,000 mm. In the Kintetsu Ikoma funicular line in Nara, Japan, both two-rail and four-rail configurations are combined in the track superstructure.

As regards funicular systems, the horizontal alignment may either be straight or it may have curved sections. The curve radii of the horizontal alignment that have been recorded are no less than R_c = 85–90 m.

Table 10.1 Number of 'funiculars' per continent and per country (2014 data)

Continent	*Country*	*Number of 'funiculars'*
Europe	Austria	13
	Azerbaijan	1
	Czech Republic	4
	Croatia	1
	France	16
	Georgia	1
	Germany	12
	Greece	1
	Hungary	1
	Italy	26
	Lithuania	2
	Norway	2
	Poland	3
	Portugal	8
	Romania	1
	Russia	3
	Slovakia	1
	Spain	11
	Sweden	2
	Switzerland	49
	Turkey	2
	Ukraine	1
	United Kingdom	18
	Bosnia-Herzegovina	1
North America	Canada	2
	United States	14
South and Central America	Argentina	1
	Brazil	8
	Chile	9
	Colombia	1
	Mexico	1
Asia	China	1
	India	1
	Israel	1
	Japan	20
	Malaysia	1
	Thailand	3
	Hong Kong	2
	Lebanon	1
Africa	South Africa	1
Oceania	New Zealand	1
Total		249

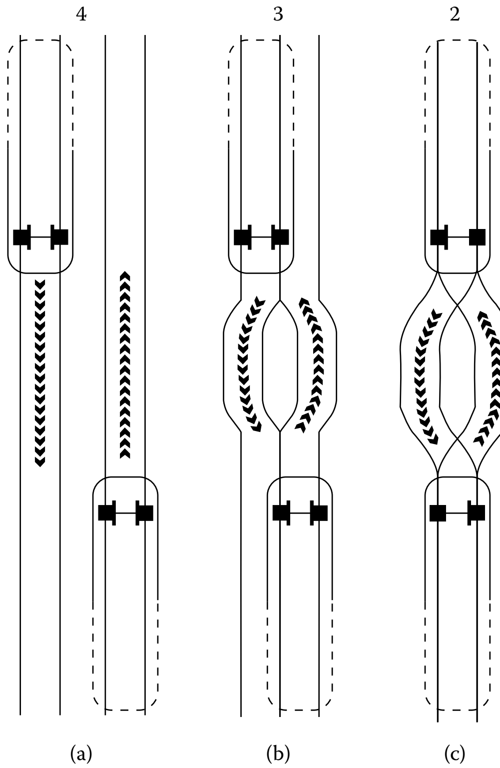

Figure 10.3 Track superstructure configurations of 'funiculars'. (a) Four-rail superstructure configuration. (b) Three-rail superstructure configuration with passing loop. (c) Two-rail superstructure configuration with passing loop. (Adapted from Cmglee at English Wikipedia. 2010, Online figure available at: http://en.wikipedia.org/wiki/Funicular.)

The three larger (in terms of length) lines are:

- Flattach Molltaler Gletscher, in Austria (S = 4,827 m)
- Sierre-Montana-Cranz, in Switzerland (S = 4,192 m)
- Pitzaler Gletscherbahn, in Switzerland (S = 3,693 m)

The three steepest lines are:

- Reina Victoria, in Chile (i_{max} = 1,200‰)
- San Cristobal, in Chile (i_{max} = 1,110‰)
- Grosslockner, in Austria (i_{max} = 950‰)

During 2016, the completion of the construction of the new funicular line, Rear Schlattli, in Switzerland which will replace the line Drahtseilbahn Schwyz Stoos is expected. This new line will have a maximum longitudinal gradient of 1,110‰ (Standseilbahnen, 2015).

The vast majority of funicular lines are constructed at grade. However, underground systems also exist (e.g., Pitzaler Gletscherbahn in Tirol, Austria; Val Gardena Ronda Express in Italy; Carmelite in Israel; Metro Alpin Saas-Fee in Switzerland).

The track superstructure can be ballasted, made on concrete or made on steel. The rails are mounted on the track superstructure with special rail fastenings at intervals of about 80 cm.

Figure 10.4 Two-rail configuration with passing loop (single-lane shuttle with Bypass), Salzburg, Austria. (Photo: A. Klonos.)

Figure 10.5 Two-rail configuration with passing loop (single-lane shuttle with Bypass), Cape Town, South Africa.

Figure 10.6 Three-rail configuration funicular, Angel flight in Los Angeles, USA. (Adapted from Sullivan, J. 2005, Online image available at: http://www.10best.com/interests/16-fantastic-funiculars/ (accessed 8 August 2015).)

Figure 10.7 Four-rail configuration funicular (dual-lane shuttle), Valparaíso, Chile. (Adapted from Bahamondez, M. 2006, Online image available at: https://commons.wikimedia.org/wiki/File:Ascensores_de_Valparaiso.jpg (accessed 8 August 2015).)

Table 10.2 Basic constructional and operational characteristics of 'funiculars'

Connection length	Usually $S < 1{,}200$ m, $S_{min} = 39$ m, $S_{max} = 4{,}827$ m
Running speed	Usually $V < 20$ km/h, $V_{min} = 3.6$ km/h, $V_{max} = 50.4$ km/h
Longitudinal gradient	Usually i = 300–500‰ i_{min} = 90‰, i_{max} = 1,200‰
Track gauge	Various (usually 2e = 1,000 mm)
Track superstructure configuration	• Single track – 2 rails – passing loop • Double track – 3 rails – passing loop • Double track – 4 rails – without passing loop
Vehicle transport capacity	10–420 passengers, usually 50–80
Traction cable gripping	Permanently attached
Number of cars	2 cars – one ascending, one descending

The total implementation cost of a funicular system varies between €15 M and €25 M/track-km (2014 data) (Audit Scotland, 2009; The Herrald, 2014).

10.2.2.2 Rolling stock

The main principle of 'funicular' operation is that the two cars are permanently attached to each other by a cable which runs through a pulley at the top of the incline (Figure 10.2). Counterbalancing of the two cars, with one ascending and one descending, minimises the energy needed to lift the ascending car. The cars can be attached to a second cable running through a pulley at the bottom of the incline in case the applied gravity force is insufficient to activate the mechanism and operate the vehicles on the slope.

'Funiculars' rank amongst the safest means of transport. This is due to the many safety devices used (Doppelmayr, no date).

Braking is ensured through

- A 'service' brake, which can bring the entire system to a standstill. It is applied in some specific cases: power failure, insufficient voltage and so on.
- A 'security' brake, which can immobilise the entire system at any point along the route. This brake is activated in case of cable sliding or slipping, violation of the vehicle's speed restriction, opening of the door while moving and so on.
- Two 'auxiliary' brakes, which are automatically applied if the hauling cable brakes, or if the vehicle's speed exceeds a certain threshold.

The vast majority of 'funiculars' use electricity as an energy source, but there still exist funiculars powered by water (e.g., Elevador do Bom Jesus, Braga, Portugal; Nerobergbahn-Wiesbaden, Germany) or mechanically (e.g., Siclau, Romania, Greenwood Forest Park, Wales).

In 2000, the world's first 'funicular' powered by solar energy was put into operation, in Italy (Montenero line) (Museo Galilleo, no date).

In 2001, the 'funicular' system Fun'ambule, Neuchatel, commenced operation in Switzerland, and it was the first system in the world to be equipped with a level compensation system. Its vehicles are composed of four articulated carriages which remain constantly in a horizontal position regardless of the change of inclination of the track (Doppelmayr, no date).

In 2007, the Hungerburgbahn line in Innsbruck, Austria, commenced operations (Figure 10.8). This system is characterised by numerous technical innovations and unique architecture.

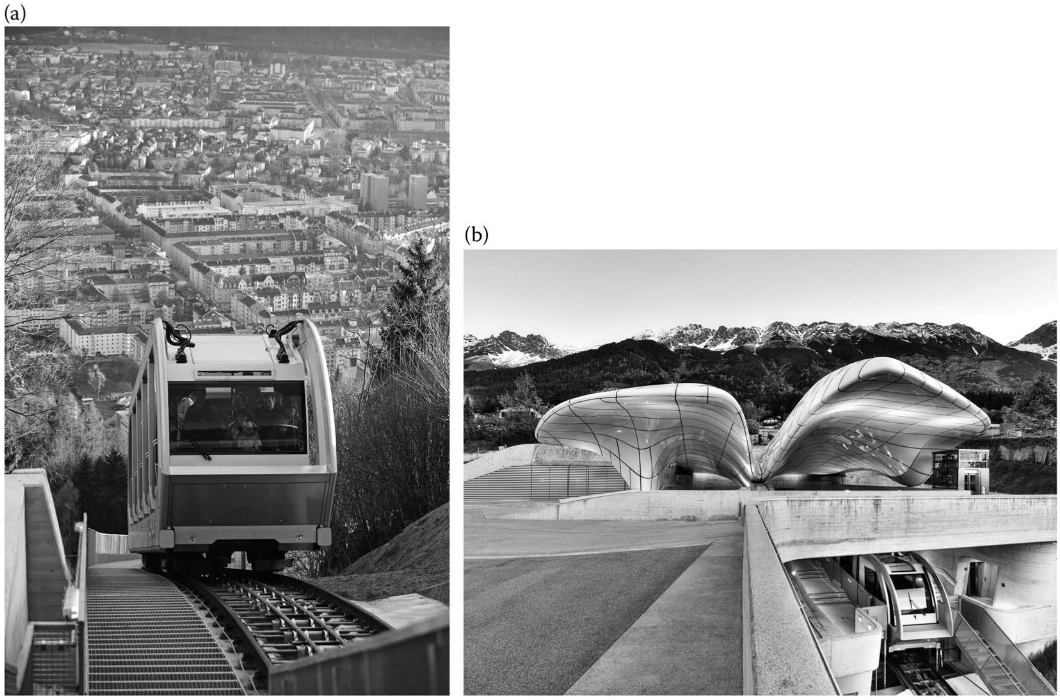

Figure 10.8 Hungerburgbahn, Innsbruck, Austria. ((a) Adapted from Innsbrucker Nordkettenbahnen Betriebs GmbH, 2008, *The Hungerburg funicular*, online, available at: http://www.nordkette.com/en/cable-railways/history/the-hungerburg-funicular.html (accessed 26 July 2012); (b) www.guentheregger.at 2013.)

10.2.2.3 Operation

Cable railway systems operate on constant speed, which, in the case of 'funiculars', ranges from 5.4 to 50.4 km/h (Wurzeralmbahn, Austria, i = 300‰). In most cases, $V < 20$ km/h. Concerning vehicle's passenger transport capacity, funiculars can accommodate up to 420 persons (Vomero – Centrale, Naples, Italy); however, the usual figures for most systems range between 50 and 80 passengers.

Transport capacity depends on the connection length S, the vehicle's running speed V, the vehicle's passenger transport capacity C_v and the dwell times at the terminal stations t_{ts}.

For a service that has no intermediate stops, and for S = 1,000 m, V = 15 km/h, $C_v = 100$ passengers and $t_{ts} = 2$ min in each terminal, the system's transport capacity results in 1,000 passengers/h/direction.

10.3 THE INCLINED ELEVATOR

The operation of inclined elevators is performed using the Shuttle 'principle' (see Section 7.2.2.1).

Regarding the track superstructure configuration, it is either single-lane shuttle or dual-lane shuttle type.

In the first case, the system operates using one single vehicle which moves between terminals, along a single track, without any passing loop, using a single pull cable. The total number of stations is two, that is, the two terminals, but it is also possible to have up to three intermediate stations.

In the second case, the system operates using two vehicles which move independently from one another on a single track each, without intersection, using a single pull cable.

It is, in essence, a double single-lane shuttle. This configuration ensures a high level of service, while off-peak, only one train may operate as a single-lane shuttle. There are only a few systems in the world that operate using this system. Some classic examples are the system of Montmartre, Paris, France (Figure 10.9), the Universeum system in Gothenburg, Sweden and the system in Spa, Belgium.

During its 1991 renovation, the Montmartre funicular railway was converted from a traditional four-rail funicular railway where the two cabins counterweigh each other, to two totally independently operated inclined lifts. This allows one cabin to remain in service if the other must be taken out of service for maintenance. The two cabins are completely automatic in operation, using the weight of the passengers as a determining factor for when it is time to depart. Therefore, especially at busy times, it can happen that both cabins will be travelling in the same direction at the same time (Wikipedia, 2015c).

Table 10.3 summarises the main constructional and operational characteristics of inclined elevators.

The longest system is the EDF, Saint Martin de Vesubie, in France ($S = 1600$ m). The length values that are commonly recorded are smaller ($S < 500$ m).

The Lärchwand-Schrägaufzug line in Austria features the largest width among all 'inclined elevator' lines in the world (width equal to 8,200 mm).

The inclined elevator of the Eiffel Tower in Paris, France, has a longitudinal gradient of 4,761‰, the system at the stadium for the olympic games in Montreal, Canada, has a longitudinal gradient of 1,880‰ while the Scenic Railway Katoomba system in the Blue Mountains, Australia, has a maximum longitudinal gradient of 1,288‰. Usually gradients vary between 500‰ and 700‰.

The recorded speeds of inclined elevators range from 1.8 to 39 km/h (Dorfbahn, Tirol, gradient 53.5‰). Usual speed values do not exceed 8 km/h. Vehicle passenger transport capacity usually ranges between 40 and 60 people, but there are cases, such as the Dorfbahn in Tirol, which can carry up to 270 passengers.

For a service without any intermediate stops, and for $S = 300$ m, $V = 8$ km/h, $C_v = 40$ passengers and $t_{ts} = 2$ min in each terminal, the system's transport capacity results in 564 passengers/h/direction.

Figure 10.9 The Montmartre funicular system in Paris, France (double inclined elevator). (Adapted from Breuer, R. 2004, Online image available at: http://en.wikipedia.org/wiki/Montmartre_Funicular,2014 (accessed 8 August 2015).)

Table 10.3 Main constructional and operational characteristics of inclined elevators

Connection length	Usually S < 500 m, S_{min} = 26 m and S_{max} = 1,600 m
Running speeds	Usually V < 10 km/h, V_{min} = 1.8 km/h and V_{max} = 39 km/h
Longitudinal gradient	Significantly high values are recorded. Usually i = 500‰–700‰
Track gauge	Various
Track superstructure configuration	Single track – 2 rails – without passing loop
Vehicle transport capacity	3–270 passengers (usually 40–60)
Traction cable gripping	Permanently attached
Number of cars	1 car

10.4 ADVANTAGES AND DISADVANTAGES OF CABLE RAILWAY SYSTEMS FOR STEEP GRADIENTS

Advantages:

- They can move on very steep gradients.
- They are environment-friendly, regarding both air and noise pollution.
- 'Funiculars' are used as a feeder transport service for urban railway networks at areas with large relief.

Disadvantages:

- They provide service for short distances only.
- Transport system capacity is rather restricted and fixed.
- Seamless transit and connection with other systems is not possible.
- System operation requires specialised facilities and rolling stock.

10.5 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

'Funiculars' are typically implemented when the distances that are to be connected do not exceed 5 km, when there are continuous and steep gradients and a relatively high traffic demand (1,000–2,000 passengers/h/direction).

'Inclined elevators' are designed for even shorter distances (<1.5 km), with continuously present and very steep gradients and relatively low transport volume (200–700 passengers/h/direction).

In hilly cities, cable-propelled railway systems are used:

- For passenger transport (in some cases they are considered as public transport systems – e.g., the Petrin funicular in Prague, Czech Republic; the St Jean – Fourvière/St Just line in Lyon, France).
- For the transport of skiers to the ski resorts.
- As 'inclined lifts' to serve hotels' guests.
- For industrial use – power station funiculars are the most commonly used ones (UK, Switzerland).
- Some of the 'funiculars/inclined elevators' constitute attractions at thematic parks themselves.

This railway transport mode has been abandoned in many countries. However, the fact that during the last 10 years, 9 new funicular systems have been put in operation while many existing ones were renovated/upgraded proves that this transport mode remains to be considered as an alternative for situations of complex landscaping.

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Chapter 11

Organisation and management of passenger interurban railway transport

11.1 SERVICES AND BASIC DESIGN PRINCIPLES

The process of the design of railway services is generally aimed at the definition of the services that will be provided to its customers (Lambropoulos, 2004).

It takes place at strategic, tactical and operational levels:

- At the strategic level (long term), alternatives are examined and decisions are taken regarding the necessary investment in infrastructure and rolling stock.
- At the tactical level (mid-term), the infrastructure and the rolling stock are considered to be decided upon and the timetables that will be offered to the customers are designed. In addition, the rolling stock providers are determined and issues concerning the staff are addressed.
- At the operational level (short term), timetables are already determined. At this level, emergencies and special incidents (e.g., long delays, equipment failures, track works, accidents and events) are treated and personnel shifts are determined.

The main objective of the passenger railway transport system design is to identify the services that will be offered to the system's potential customers in terms of timetables (scheduled services) and additional services.

The following services are offered in passenger railway transport (Samuel, 1963; Alias, 1985; Lambropoulos, 2004):

- Urban services (tramway, metro, monorail, low and medium transport capacity railway systems)
- Suburban services
- Regional services
- Long-distance services of conventional speed (conventional or tilting trains)
- Long-distance services of high speed (high-speed conventional or tilting trains)
- International passenger services (conventional-speed or high-speed trains)
- Local services
- Miscellaneous services (airport links, touristic)

In most of the above cases, the railway infrastructure facilitates more than one of the aforementioned services. A typical example of this is the fact that the same track can accommodate suburban, regional and interurban trains. The infrastructure manager of such a mixed traffic composition passenger network must organise a combination of services in the provided infrastructure in a way that all services are supplementing each other while at the same time avoiding any potential conflict.

As regards the design of passenger railway transportation, the following parameters are mainly considered in its definition (Lambropoulos, 2004):

- The type of services offered by the railway system to its potential customers in terms of scheduled services (timetables) and added services
- The origin and destination stations of itineraries as well as the intermediate stations
- The departure/arrival times of trains at terminals and intermediate stations
- The service frequency
- The type of the rolling stock to be used and the train formation
- Commercial issues such as fares and their structure

In particular, for timetable construction, it is essential that travel times of trains between stations are known in advance. These travel times are either calculated (using simulation models) or measured using a chronometer. The hierarchy of trains, namely the classification of trains based on their priority, also plays an important role. Trains of higher priority are given priority at a track block section over trains of lower priority.

The timetabling procedure for passenger trains may regard

- A system that is already in operation. This case requires either adjusting to any probable demand changes, or offering new 'products' to its clientele.
- A brand new railway link under construction. In this case the target is to ensure the best choices for infrastructure and rolling stock so that the system will
  - Offer a high level of service to its users
  - Be economically viable for the operator
  - Respect the environment

The following sections provide an analysis of the long-distance services only as all the other services have either been already analysed or will be analysed in other chapters.

11.2 SERVICE LEVEL OF INTERURBAN PASSENGER RAILWAY TRANSPORT: QUALITY PARAMETERS

The assessment of the quality of service for an intercity passenger train service is based on the degree of satisfying the following quality parameters:

- Short travel time and, most importantly, competitive with the travel time of other means of transport
- Reliability and regularity of scheduled services
- Service frequency
- Relatively low and competitive fares
- 'Dynamic comfort' for passengers during transportation
- Comfort while travelling (ample legroom and space, cleanliness and aesthetics of trains, air conditioning)
- Security during transportation
- Security at railway stations
- Ease in issuing and purchasing tickets
- Special services provision, for example, services for accompanying private cars, night trains, restaurant, cafeteria and other similar services

For international passenger services, two additional parameters have to be considered:

- Minimise delays at border stations
- Ensuring interoperability

The basic criterion for assessing the above parameters is their comparison with the relative parameters of other competitive means of transport.

The conventional-speed interurban services involve connections of large urban centres located within a distance of more than 150 km but of an ability to reach 1,500 km (when no train change is involved). The maximum running speeds are V_{max} = 160–200 km/h, whereas commercial speeds are V_c < 150 km/h. Intercity trains are usually scheduled to run every 1–2 h. In these cases, trains have rather limited stops (the average distance ranges from 50 to 100 km), but much emphasis is put on speed and mostly on passengers' comfort (dynamic, space) (Jorgensen and Sorenson, 1997). The halt time of a passenger train at a station is contingent on a series of factors such as the construction characteristics of platforms and trains, the number of passengers boarding and alighting at each station, etc. This time span is usually around 1–2 min.

The high-speed interurban services involve connections of large urban centres and mainly connections between both edges of high-volume 'bi-pole trips'. They usually serve distances bigger than 300–400 km, the average being around 500 km (e.g., Paris–Lyon, Tokyo–Osaka and Frankfurt–Hamburg) reaching more than 1,500 km (maximum distance is recorded in China: 2,300 km, Beijing–Guangzhou). In these journeys, stops are limited (the average distance varies between 150 and 250 km); however, the choice of stopping points depends on the characteristics of the connection. When choosing the stopping points, the train operator must balance the cost of the stop (lost time, increased energy consumption for acceleration) and the potential gain in passenger volume. Particularly strong emphasis is placed on reliability and punctuality of scheduled services. The maximum running speeds that trains can achieve are V_{max} = 200–320(350) km/h, while commercial speeds are V_c > 150 km/h (see Chapter 12). High-speed trains are scheduled to run at intervals of 1–2 h. The rolling stock that is scheduled for a connection is specially designed and manufactured, as is the railway infrastructure and the signalling system.

International – high and conventional speed – trains fall into a special category of interurban trains. These trains must abide by interoperability standards. Emphasis is particularly placed in facilitating border crossing and in minimising delays at borders.

Regardless of the category of services offered in terms of speed, it is essential that the running speed remains constant during the biggest part of the route in order to secure a wisely managed and correctly operating interurban railway network.

11.3 SCHEDULING OF PASSENGER TRAIN SERVICES

In most railway corridors, the system of clockface timetable has been in use for many years. Trains run at regular and quite dense time intervals (e.g., 6:12, 7:12 and 8:12). This system ensures that timetables are easily memorised by the users and that the railway establishes itself as a reliable system of round-the-clock services available to the user. However, sometimes services are less frequent outside the peak hours.

Nowadays there is a tendency for organising an interconnecting network of services that run on clockface timetable (Lambropoulos, 2004). In this context

- The main corridors where fast trains move at rhythmic headway of 1 h ('mainstream' services) are determined.

- The corridors where feeding trains move rhythmically toward the main axis corridors are determined.
- At junction stations, it is ensured that the trains of one branch correspond to those of the other.
- It is also ensured that trains arrive in such a way that connections with and transfers to all other lines and destinations become feasible.

This system has been coded as integrated clockface timetable and is widely used today in Switzerland, Austria, the Netherlands and, partly, in Germany ('Inntegraler Taktfahrplan').

It should be noted here that the various categories of railway services as well as the differentiation between them is sometimes quite difficult to discern (there are, for instance, systems and services that are found both in metros and suburban trains, in regional and interurban trains, etc.)

The passenger train services are provided on a constant basis according to itineraries which are announced in advance. This is something that is taken for granted with regard to passenger services as opposed to freight ones.

11.4 SELECTION AND PURCHASE OF ROLLING STOCK

Rolling stock procurement must be based on a thorough study of the needs and, generally, of the railway system's 'environment'.

In particular, the purchase of new rolling stock must allow the system operator to

- Renew the rolling stock
- Increase the quality of the provided passenger and freight services and be prepared to respond to any increase in the transportation demand
- Cover any shortage in rolling stock or replace technically defective rolling stock the reparation of which is not economically viable
- Take advantage, in the best possible way, of the new possibilities offered by infrastructure that is already completed or is expected to be completed in the near future
- Increase the competitiveness of the railway services over other means of transportation (land, air) and claim a larger share in the passenger transportation market
- Respond to interoperability needs
- Increase the safety level of train traffic

Significant factors that influence the design process of the procurement are

- The useful lifespan of rolling stock
- The development of the railway infrastructure
- The type of services provided

As regards the first factor, the viability of all railway vehicles, according to international practice, is considered to range between 30 and 35 years, given that maintenance and repair work of vehicles is carried out according to the approved specifications (Baumgartner, 2001). A reconstructed vehicle is considered to have a 15-year useful lifespan on completion of its reconstruction.

The railway infrastructure comprises the track as well as all the civil engineering structures, systems, facilities and premises ensuring the train traffic. The constructional characteristics of the aforementioned components must allow the running of the trains at the track design speed (V_d), whereas the track maintenance and other predictable or non-predictable parameters (level crossings, etc.) define the permissible track speed (V_{maxtr}). The objective of the infrastructure manager is to ensure that the permissible track speed be close to the track design speed. As regards the design speed of the rolling stock V_{rs} moving on the railway infrastructure, if major upgrades of infrastructure are not foreseen during its useful lifespan, it should be equal to or slightly higher than the track design speed.

The methodology that is used so as to calculate the necessary rolling stock that needs to be acquired for a specific network is described in the following. The steps that ought to be followed are briefly illustrated in the flowchart presented in Figure 11.1 (Pyrgidis, 2010). To better understand this methodological approach, an example concerning the case of dedicated passenger traffic operation with different offered services (interurban, suburban, and regional) is also cited (see Figure 11.2).

11.4.1 Step 1: Assessment of the existing situation

This step includes gathering of specific data related to the railway infrastructure, to the existing available rolling stock and the train timetable schedule.

Concerning the railway infrastructure, this step involves recording the number of tracks (double and single), the traffic mix (whether passenger and freight trains run on the network), the traction system, the track design speed, the civil engineering structures (tunnels, bridges, overpasses, etc.), the lineside systems (level crossings, electrification, signalling and telecommunication systems) and the facilities and premises (stations, depots, etc.) for each line of the network (see Table 11.5, column 2).

Concerning the existing available railway rolling stock, this step involves recording the elements featured in Table 11.1, as per category of rolling stock.

The assessment of the existing available railway rolling stock is followed by the recording of the passenger and freight service connections of the network.

The elements that are necessary for a full recording of passenger service connections are indicated in Table 11.2. In this table, a very small radial-shaped railway network of three corridors (A–B, A–C, and A–D) is considered as an example (Figure 11.2). This network is exclusively used by passenger trains.

The processing of the timetable provides the elements related to the service frequency to each connection (Table 11.3).

Finally, Table 11.4 indicates a hypothetical scenario of an existing situation of the rolling stock that satisfies the service conditions of passenger trains considered in Table 11.2.

In the case of freight traffic, additional data regarding the train type in relation to its service frequency (regular, periodic, and optional), the train weight, the train type in relation to the load processing (e.g., unit train) the wagon type per train, etc. are collected.

11.4.2 Step 2: Determination of the target year

Once the data of the existing available rolling stock are recorded, the target year must be determined in order to evaluate the rolling stock needs; this target can be immediate (4 years), mid-term (10 years) or long term (20 years).

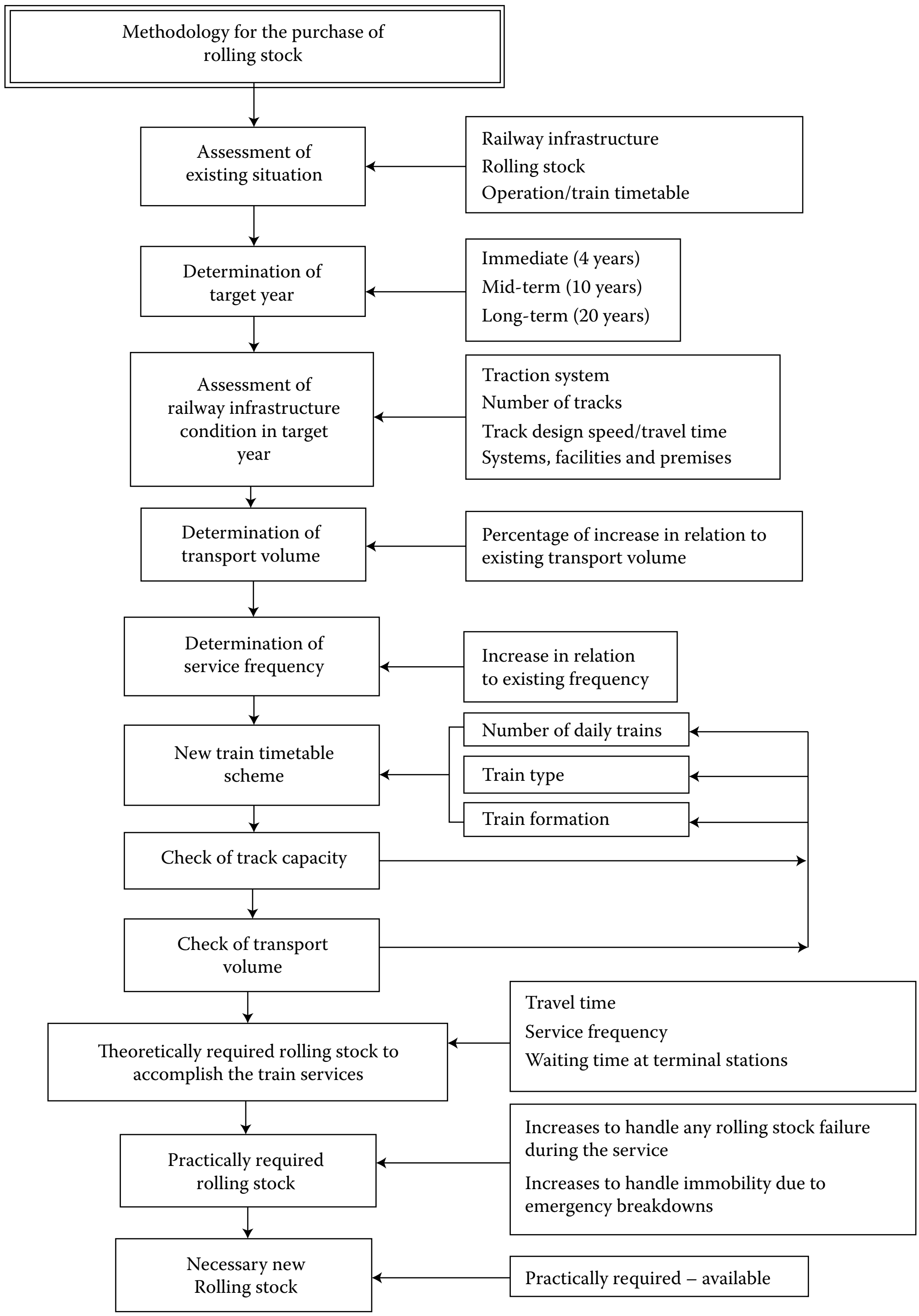

Figure 11.1 Methodological approach – steps for the calculation of the purchase of rolling stock. (Adapted from Pyrgidis, Ch. 2010, Estimation process of future rolling stock needs for a railway network, *5th International Congress on 'Transportation Research in Greece'*, 27–28 September 2010, Volos, Congress Proceedings, CD.)

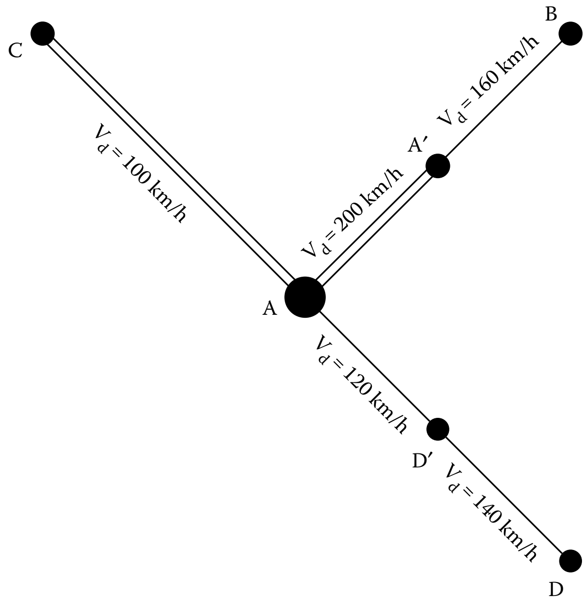

Figure 11.2 Configuration of railway network considered – existing situation.

11.4.3 Step 3: Assessment of the situation in the target year

The determination of the target year is accompanied by the adoption of a scenario related to the completion of the ongoing or planned works, which aim at the improvement of the railway infrastructure. This procedure is perhaps the most important one in terms of successful evaluation of the needs in rolling stock. It should also be highlighted that on most

Table 11.1 Assessment of the existing condition of the rolling stock

Elements	*Loco motives*	*Railcars/MUs*	*Passenger cars*	*Freight wagons*	*Shunting locomotives*
Model/manufacture series	√	√	√	√	√
Country of origin/manufacturer	√	√	√	√	√
Traction system	√	√			√
Number of units in operation	√	√	√	√	√
Nature of procurement	√	√	√	√	√
Year of first circulation	√	√	√	√	√
Design speed	√	√	√	√	√
Axle load	√	√	√	√	√
Total motor power	√	√			√
Transport capacity (seats)		√	√		
Dedicated corridor	√	√	√	√	
Description/formation		√			
Interior configuration			√		
Characterisation in terms of operability of the vehicle			√	√	

Table 11.2 Data recorded for each service – hypothetical indicative example

Origin–destination (connection)	*Train category (service provided)*	*Type of train*	*Traction system*	*Train formation*	*Travel time*	*Daily trains per direction*	*Seats per train*	V_{maxtr} *(km/h)*	V_{rs} *(km/h)*
A–B/B–A	Interurban (intercity) express	Loco-hauled	Diesel	1LC + 4TC	4 h 15 min	3/3	252	160	200 (LC) 160 (TC)
A–B/B–A	Interurban (intercity)	Railcar	Diesel	1PR + 3TC′ + 1PR	5 h	5/5	220	160	160
A–C/C–A	Suburban	Railcar	Electric	1PR + 2TC′ + 1PR	1 h	16/16	180 (190 standing)	100	120
A–D/D–A	Regional	Railcar	Diesel	1PR + 1PR	1 h 30 min	5/5	144	140	160
A–D/D–A	Regional	Loco-hauled	Diesel	1LC + 8TC	1 h 50 min	3/3	504	140	160 (LC) 160 (TC)

LC: Locomotive.
PR: Single railcar.
TC: Trailer vehicle of a loco-hauled train.
TC′: Trailer vehicle of a railcar.

Table 11.3 Service frequency as a result of Table 11.2 – hypothetical indicative example

Connection	*Daily trains per direction*	*Service frequency (line operating 16 h daily)*
A–B/B–A	8/8	Every 2 h
A–C/C–A	16/16	Every 1 h
A–D/D–A	8/8	Every 2 h

Table 11.4 Current situation of the rolling stock – hypothetical indicative example – hypothetical scenario

Category/type/ formation of trains	*Number of units in operation*	*Age (years)*	*Design speed V_{rs} (km/h)*	*Train transport capacity (seats)*
Diesel railcars (1PR + 3TC′ + 1PR)	6	10	160	220
Diesel railcars (1PR + 1PR)	4	5	160	144
Electric railcars (1PR + 2TC′ + 1PR)	5	10	120	180 + 190 standing
Diesel locomotives (LC)	4	2	200	–
Diesel locomotives (LC)	4	20	160	–
Passenger cars (TC)	40	15	160	63

occasions the new works are those that guide and set the requirements for the purchase of the new rolling stock. The whole network is divided into tracks and into track sections and the works implementation scenario includes, for each separate track (or track section) of the network, the following elements:

- The traction system (diesel traction or electrification)
- The number of tracks (single or double track)
- The track design speed and the anticipated connection travel time

The hypothetical scenario may also include the configuration of new elements for facilities and premises expected to have a significant impact on the availability of the rolling stock and on the provided freight transport services, and thus on the setting up of the investment programme for the procurement of the rolling stock. Examples of such facilities and premises are the stations and the depots.

In the hypothetical example presented in this section, mid-term planning is being considered (after 10 years, see Table 11.5).

11.4.4 Step 4: Determination of the transport volume target

At this stage, an assumption of increase of the passenger transport volume is made (based on the results of an appropriate feasibility study) and the percentage of variation of such volume is defined. In the hypothetical example presented in this section, a 60% increase of all connections is considered (see Table 11.6). Thus, this creates a need for a redefinition of the train timetables and for the elaboration of a new train timetable scheme.

Table 11.5 Rail infrastructure conditions in the target year – hypothetical indicative example – hypothetical scenario

	Existing situation (year 2014)	*Year 2024 (target year, medium-term approach)*
(1)	*(2)*	*(3)*
Corridor A–B Section A–A′	Double track Diesel traction V_d: 200 km/h V_{maxtr}: 160 km/h (due to the absence of electrification) Minimum travel time: t = 2 h 15 min	Double track Electrification V_d: 200 km/h V_{maxtr}: 200 km/h Minimum travel time: t = 2 h
Corridor A–B Section A′–B	Single track Diesel traction V_d: 160 km/h V_{maxtr}: 160 km/h Minimum travel time: t = 2 h	Double track Electrification V_d: 200 km/h V_{maxtr}: 200 km/h Minimum travel time: t = 1 h 30 min
Corridor A–C	Double track Electrification V_d: 100 km/h V_{maxtr}: 100 km/h Minimum travel time: t = 1 h	Double track Electrification V_d: 140 km/h V_{maxtr}: 140 km/h Minimum travel time: t = 45 min
Corridor A–D Section A–D′	Single track Diesel traction V_d: 120 km/h V_{maxtr}: 120 km/h Minimum travel time: t = 1 h	Double track Diesel traction V_d: 160 km/h V_{maxtr}: 120 km/h Minimum travel time: t = 45 min
Corridor A–D Section D′–D	Single track Diesel traction V_d: 140 km/h V_{maxtr}: 140 km/h Minimum travel time: t = 30 min	Double track Electrification V_d: 160 km/h V_{maxtr}: 160 km/h Minimum travel time: t = 25 min

Table 11.6 Hypothetical indicative example – estimation of the number of passengers that must be served by the new routing design for the target year

Existing situation	Maximum number of users *that can be served* daily, per connection per direction A–B: 252 × 3 = 756 A–B: 220 × 5 = 1,100 A–C: 370 × 16 = 5,920 A–D: 144 × 5 = 720 A–D: 504 × 3 = 1,512 10,008 passengers daily in total (one direction) Number of users *that are served* daily per connection per direction (hypothetic) A–B: 1,600 (<1,856) A–C: 5,300 (<5,920) A–D: 2,100 (<2,232) 9,000 passengers in total (one direction) < 10,008
Target year	Maximum number of users *that have to be served*, daily per connection per direction 14,400 per day in total (one) direction (increase 60%) A–B: 2,560 A–C: 8,480 A–D: 3,360

11.4.5 Step 5: Determination of the service frequency target

The next step consists of the determination of the service frequency target and, in particular, of the increase in service frequency in relation to the existing frequency. The frequency is related to the level of service that the railway company wishes to offer to its customers (see the example in Table 11.7). The increase of speeds, which entails a reduction of the travel time, has a positive impact on such a change.

11.4.6 Step 6: New train timetable scheme

Having taken into consideration all the previous steps, a new train timetable scheme is set for the target year. In this plan, an indicative example of which is given in Table 11.8, the following data are recorded for each connection:

- The train category (services provided)
- The type of train in terms of traction and formation
- The train transport capacity (seated passengers for interurban and regional services, seated and standing passengers for suburban services)
- The maximum running speed V_{max} (between stations) ($V_{maxtr} = V_{max}$)
- The travel time
- The number of accomplished services per day

Some of the basic principles that must be followed during the design of the new timetable scheme are

- Efficient utilisation of the railway infrastructure
- Exclusive routing of electric trains on tracks electrified for the total of their length

Table 11.7 Hypothetical indicative example – target year – desired service frequency

Connection	*Daily trains per direction*	*Service frequency (track operating 16 h daily)*
A–B/B–A	16/16	Every 1 h
A–C/C–A	24/24	Every 40 min
A–D/D–A	16/16	Every 1 h

Table 11.8 Hypothetical indicative example – scenario: Passenger trains scheduled for the target year

Connection (origin–destination)	*Train category (services provided)*	*Type of train in terms of traction and formation*	*Seats per train*	*Maximum running speed (km/h)*	*Travel time*	*Daily trains per direction*
A–B/B–A	Interurban Express	Electric loco-hauled trains (1LC + 5TC)	315 = 5 × 63	200	3 h 30 min	8/8
	Interurban	Electric railcars (1PR + 2TC′ + 1PR)	180		4 h	8/8
A–C/C–A	Suburban	Electric railcars (1PR + 4TC′ + 1PR)	320 (320 standing)	140	45 min	24/24
A–D/D–A	Regional	Diesel loco-hauled trains (1LC + 5TC)	315 = 5 × 63	160	1 h 10 min	16/16

- Routing of diesel trains at connections where only certain track sections are electrified and no stops at major hub stations are included
- Use of interchange only when the connection to the final destination is direct and of suburban nature
- The design speed V_{rs} of the new trailer vehicles must be equal to the maximum track design speed V_d applied in the network (in our case V_{dmax} = 200 km/h)

11.4.7 Step 7: Checks on corridor track capacity and transport volume

After the elaboration of the new timetable scheme, checks are conducted with regard to track capacity and transport volume.

The percentage of availability of each corridor in relation to its track capacity should be greater than or equal to 25%. Should the new service schedule not satisfy the above criterion, appropriate interventions need to be made. These relate to an adjustment of the number of trains, of the type of trains as well as of their formation aiming at a correspondence between the existent corridor track capacity and the new timetable scheme.

Checking the transport volume includes checking the accomplishment of the transport volume that was previously defined as target. Having calculated the total availability of passenger seats with an assumption of an average occupancy of passenger trains of 70%, one must examine if the timetable scheme adopted satisfies the achievement of the transport volume target at the forecast period of time.* Should the result of the control be negative, appropriate changes are made related, as in the case of the track capacity, to interventions in the number of trains, to the type of trains and to their formation.

In our case study, the transport volume check is positive.

Transport volume check

A–B: $[(8 \times 5 \times 63) + (8 \times 180)] \times 0.70 = 2{,}772 > 2{,}560$
A–C: $(24 \times 620) \times 0.70 = 10{,}752 > 8{,}480$
A–D: $(16 \times 5 \times 63) \times 0.70 = 3{,}528 > 3{,}360$

11.4.8 Step 8: In theory: Required rolling stock for the performance of scheduled services

To calculate the theoretically required rolling stock for the performance of the train services, the following factors are taken into consideration:

- Travel time
- Service frequency
- Waiting time of trains at the terminal stations

Required number of locomotives

1. *Connection A–B/B–A, electric loco-hauled trains:* The travel time of electric loco-hauled trains is equal to 3 h and 30 min (one-way trip). Hence, for a complete itinerary (round trip), a total of 7 h are required. The waiting time at each terminal is considered to be equal to 1 h. This time period is practically implying the realisation of intercity services every hour (at hourly intervals).

* In the case of suburban trains, the number of standing passengers is also recorded.

Therefore, the total time that is required by a train in order to run a full service is $3.5 + 3.5 + 1 + 1 = 9$ h. On the basis of the new timetable scheme, the service frequency operated by the electric loco-hauled trains is 2 h. The number of electric locomotives theoretically required for the performance of services is calculated: 9 h/2 h = 4.5. This number is rounded to the next highest integer (i.e. 5).

2. *Connection A–D/D–A, regional services, and diesel loco-hauled trains:* The travel time of diesel loco-hauled trains is equal to 70 min. The waiting time is considered to be equal to 50 min. Therefore, the total time that is required by a train in order to run a full service is $1.166 + 1.166 + 0.833 + 0.833 = 3.998$ h. On the basis of the new timetable scheme, the service frequency operated by the diesel loco-hauled trains is 1 h. Thus, as in the previous case, 3.998 h/1 h = 3.998 and consequently four diesel locomotives are required.

Required number of railcars

1. *Connection A–B/B–A, long-distance (interurban) services – electric railcars:* The total time that is needed by a train in order to run a full itinerary is $4 + 4 + 1 + 1 = 10$ h. On the basis of the new timetable scheme, the service frequency operated by the electric railcars is 2 h. Thus, as in the previous case, 10 h/2 h = 5 and consequently five electric railcars are required.
2. *Connection A–C/C–A, suburban services – electric railcars:* Given that the provided services are suburban, the waiting time at each terminal is considered to be equal to 30 min. The total time that is needed by a train in order to run a full service is $0.75 + 0.75 + 0.5 + 0.5 = 2.5$ h. On the basis of the new timetable scheme, the service frequency operated by the electric railcars is 0.666 h. Thus, as in the previous case, 2.5 h/0.666 h = 3.75 and consequently four electric railcars are required (these calculations do not take into account any additional trains needed to serve peak hours).

Required number of passenger cars

The total number of passenger cars is $5 \times 5 + 4 \times 5 = 45$.

11.4.9 Step 9: Practically required rolling stock

First of all, it must be assumed that, in the target year, all railway vehicles in service shall have an age of 30 years or less (useful lifespan). Moreover, the number of calculated locomotives and railcars shall be increased (Baumgartner, 2001):

- At first by 10% in order to take into account the existence of spare locomotives/railcars at nodal points of the network so as to be ready to handle any rolling stock failure during the service.
- Second by 20% in order to take into account the immobility resulting from the need to keep a train in the depot for repair due to emergency breakdowns.

Apart from the number of locomotives and railcars, an increase of the passenger cars by 20% is also required, in order to take into account their immobility in the depot for emergency breakdowns.

Concerning freight transport, a scenario usually based on the variation of the freight transport volume is adopted to estimate the freight wagons. This variation may relate to

- Increase of the transport volume
- Change of the transported cargo type and the offered services

- Change of the freight transport connections
- Combinations of the above

The increase of the transport volume is particularly boosted by improvement of the level of provided services and, more specifically, of the following quality parameters:

- Reduction of route travel time
- Reliability and regularity of train services
- Frequency of train services
- Safety against theft
- Competitive fares in comparison with the fares of other transport means
- Offering of special services

The suggested percentages for additions to the number of calculated locomotives are the same as for the case of passenger transport. Additional reserves are necessary due to the fact that freight wagons are subject to periodic inspection, which results in an important part of the fleet permanently undergoing inspection/repair. Moreover, in order to take into account traffic peak periods, the theoretically required freight wagons is increased by a small percentage.

Table 11.9 Hypothetical indicative example – calculation of the required rolling stock for the target year

Category/type/formation of the required rolling stock (1)	*Theoretically required (2)*	*Practically required (3)*	*Available (4)*	*Necessary (5)*	*Comments (6)*
Electric locomotives $V_{rs} = 200$ km/h	5	8	0	8	
Diesel locomotives $V_{rs} \geq 160$ km/h	4	6	4 $V_{rs} = 200$ km/h	2	The other four locomotives in operation will be 30 years old The traction characteristics of the available rolling stock also need to be checked
Electric railcars (1PR + 2TC′ + 1PR) $V_{rs} = 200$ km/h	5	8	0	8	
Electric railcars (1PR + 4TC′ + 1PR) $V_{rs} = 140$ km/h	4	6	0	6	The existing rolling stock can also be used but its design speed (120 km/h) does not satisfy the performances of the upgraded track (140 km/h)
Passenger cars $V_{rs} = 200$ km/h	45	60	0	60	The existing rolling stock will be old. Due to its low design speed (160 km/h) it can be used only on the connection A–D

11.4.10 Step 10: Required rolling stock

Having estimated the practically required rolling stock (Table 11.9, column 3, i.e. theoretically required plus increases), the necessary rolling stock is estimated for the target year (Table 11.9, column 5).

It is estimated by subtracting the available rolling stock in the target year (Table 11.9, column 4) from the practically required rolling stock.

Therefore,

Necessary rolling stock = practically required rolling stock – available rolling stock.

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Chapter 12

High-speed trains

12.1 DISTINCTION BETWEEN HIGH SPEEDS AND CONVENTIONAL SPEEDS

There is no explicit definition of the term high-speed rail.

A speed of V = 200 km/h was initially established as a limit of distinction between a train running at 'conventional' speed and a train running at 'high' speed. The main reasons for the adoption of the above limit were the following:

- In most upgraded tracks, the radii of curvature in the horizontal alignment of the layout had been selected for maximum passage speed V_{pmax} = 200 km/h.
- Just over this speed, the impacts from the geometric track defects are intensified, while some of the train functions become troublesome, and need special handling (e.g., an increase of the braking distance and the aerodynamic resistances of the trains and the train driver's inability to identify indications of the trackside signalling).

In the Technical Specifications for Interoperability (TSIs), the Trans-European High-Speed Network lines, are classified into the following three categories (EC, 1996; UIC, 2014a):

- *Category I:* New tracks that are specially constructed for high speeds and are suitably equipped, so that a running speed of $V_{max} \geq 250$ km/h can be reached.
  In some sections of these tracks, where, for technical reasons, the maximum speed provided for the interoperable trains may not be reached, it is possible for a lower permissible track speed to be imposed.
- *Category II:* Existing tracks that are specially upgraded for high speeds, and suitably equipped so that running speeds in the region of V_{max} = 200 km/h can be reached.
- *Category III:* Tracks that are specially upgraded for high speeds (V_{max} = 200 km/h), with special specifications, due to the limitations/enforcements imposed by the landscape or the compulsory passage through the urban environment, resulting in speed adjustment, depending on the case.

A third categorisation is proposed in the literature reference (Demiridis and Pyrgidis, 2012) according to which, for the distinction of a conventional-speed line from a high-speed line to take place, two conditions are used, which should be satisfied simultaneously:

1. Maximum attainable running speed of trains $V_{max} \geq 200$ km/h
2. Average running speed between two successive intermediate stations $V_{ar} \geq 150$ km/h

According to 2015 data, 21 railway networks worldwide and, more specifically, the networks of China, Spain, Japan, France, Italy, Germany, Turkey, South Korea, Taiwan, Belgium, Netherlands, Russia, United Kingdom, Sweden, Switzerland, United States, Uzbekistan, Austria, Finland, Norway, and Portugal have at least one line that satisfies the above two conditions (Hartill, 2013, 2015).

However the quality of a railway corridor's infrastructure, with regard to speed, depends on the value of the average running speed V_{ar} considering all track sections of the corridor. High-speed intercity rail services usually serve distances of more than 400–500 km. For these trips, the intermediate stops are normally very few (from 0 to up to 2 stops).

Given the above, for a distinction between a conventional-speed line and a high-speed line, that is based on the total length of a corridor, one could use as the second criterion one of the following:

- The average running speed between two successive intermediate stops. But in this case the value of the distance between two successive intermediate stops L_{st} has to be near the average distance that is used in high-speed networks.
- The commercial speed across the corridor. In this case for V_c a minimum value of 150 km/h is proposed. This value is based on the competition between train and aeroplane, in order to assure nearly equal travel times for the route.

Considering all the above, in this chapter, in order to distinguish a conventional-speed line from a high-speed line, the following three conditions that must be met simultaneously are used:

1. Maximum achievable train running speed: $V_{max} \geq 200$ km/h
2. Average running speed between two successive intermediate stops: $V_{ar} \geq 150$ km/h
3. Minimum distance between the two successive stops in which the above average speed is achieved: $L_{st} = 100$ km

On the basis of 2015 data, the first 17 out of the 21 railway networks mentioned above have at least one line that satisfies the above three conditions (UIC, 2014b; Hartill, 2015).

Finally, the first 11 networks have lines with $V_{max} \geq 250$ km/h and $V_{ar} \geq 200$ km/h. These networks can be characterised as 'very high-speed' networks (UIC, 2014a).

12.2 HIGH-SPEED TRAIN ISSUES

The increase of speed beyond a specific value creates a series of issues and possible problems, the handling of which requires special interventions regarding both the rolling stock and the track.

The systematic operation of high-speed trains in the last 30 years allowed clear identification of these problems and in some cases the setting of crucial speed limits, beyond which they arise. The basic problems caused by the development of high speeds are (La vie du rail, 1989; Pyrgidis, 1993, 1994; Profillidis, 2014) the following:

- Increase of the train's aerodynamic resistance
- Problems arising in tunnels
- Dysfunction of the trackside signalling
- Increase of the braking distance of the train
- Requirements for high tractive power
- Lateral instability of vehicles in straight paths
- Special requirements in the track geometry, horizontal and vertical alignment of design
- Noise pollution of the surrounding environment

- Severity of damage in case of an accident (collision or derailment)
- Increase of vertical dynamic loads
- Decrease in the dynamic comfort of passengers
- Troublesome passage over switches and crossings
- Intensity of the aerodynamic effects and their impacts during the movement of trains in the 'open' track, and their passage from the station platforms

It is characteristic that the above problems increase proportionally and non-linearly to the speed, resulting, beyond a specific limit, in the development of prohibitive conditions for the conventional railway.

The causes that will determine the maximum speed in the future, which may not be outperformed by the wheel–rail system, should be sought in these problems (and mainly in the increase of the aerodynamic resistances and the braking distance).

The above problems are described and discussed in the following:

- *Increase in the aerodynamic resistances of the train:* The resistance W_m of a train moving at a constant speed V on a straight path without longitudinal slopes is expressed by the following equation (the Davis equation) (Metzler, 1981):

$$W_m = A_w + B_w V + C_w V^2 \qquad (12.1)$$

where A_w, B_w, C_w: parameters depending on the characteristics of the rolling stock.

The first term, A_w, is independent of the speed of the train and represents the rolling resistances. The second term, B_wV, is proportional to the speed, and represents the various mechanical resistances (rotation of the axles, transmission of movement, etc.), as well as the air friction resistances along the train's lateral surface. The third term, C_wV^2, changes in proportion to the square of speed, and represents the aerodynamic resistances (aerodynamic drag).

According to Figure 12.1, a speed change from 200 to 300 km/h results in a change of the aerodynamic resistance of the train by 100%, while the mechanical resistances remain literally unchanged. At high speeds, the aerodynamic resistances determine,

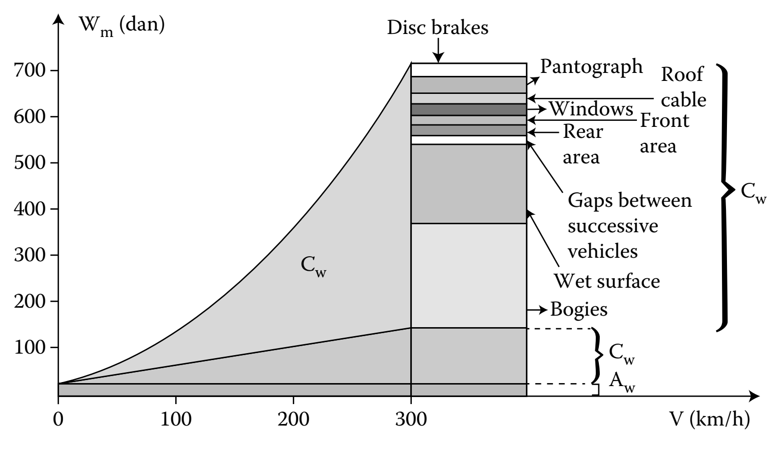


Figure 12.1 Change in the resistances of movement with respect to speed V (TGV Train A, formation: 1 power vehicle + 10 trailer vehicles + 1 power vehicle). (Adapted from Profillidis, V.A. 2014, *Railway Management and Engineering*, Ashgate; La vie du rail 1989, L'Atlantique à 300 km/h, Octobre.)

therefore, the total resistance of the train, and hence, the required motor power of the power vehicles.

- *Problems arising in tunnels:* During the passage of a high-speed train ($V_p \geq 200$ km/h) through a tunnel, the following aerodynamic problems arise (Maeda, 1996; Profillidis, 2014):
  - *Sudden change of pressure:* The passage of a train through a tunnel is always accompanied by a fluctuation in the pressure exerted frontally and laterally to the train. This fluctuation is more annoying to the passengers, as the time, within which it takes place, gets less, and therefore, as the speed of the train's movement gets higher. The great differences in the pressure inside the tunnel, and also at its entry and exit, may cause earache and headache to the passengers.

    Experimental tests have shown that, for speeds of more than 200 km/h, a notable reduction of the acoustic comfort of passengers is observed. It is mentioned indicatively that the TSI that relate to the construction and operation of the railway tunnels focus mainly on the maximum permissible change in pressure (ΔP_{max}) generated inside the tunnels. In this context, it is required that the maximum change (ΔP_{max}) in the pressure along an interoperable train should not exceed 1,000 Pa during the crossing of the tunnel, and for the maximum speed permitted by the specific civil engineering structure.
  - *High aerodynamic resistances:* During the passage of a train through a tunnel, the aerodynamic resistances, for the same speed and the same train formation, are much higher than those generated at the surface sections of the track. The major impacts resulting from the increase in the aerodynamic resistances inside a tunnel are the increase of the exerted forces on the train, which result in greater energy consumption.
  - *Interaction of trains travelling in opposite directions:* In case of a double-track tunnel, the crossing of trains travelling at speeds of more than 220 km/h may cause damage to the rolling stock (particularly breaking of window panes), due to the increased pressure waves that are generated.

    'Tunnel boom', is a phenomenon of radiation of impulsive sound from the exit of a tunnel used by high-speed trains (https://en.wikipedia.org/wiki/Tunnel, 2015).

    Upon the entrance of the train to the tunnel, shock waves are emitted from the inlet to the outer environment (entrance waves), while a similar effect is observed upon the exit of the train from the tunnel (exit waves).
- *Dysfunction of the trackside signalling:* When the speed of the train increases, the visual perception of the trackside signalling becomes increasingly difficult. In bad weather conditions (e.g., fog), the identification of signals at speeds just over 220 km/h is troublesome, if not impossible. Therefore, at high speeds, all the signalling systems that are based exclusively on the identification of signals by the train drivers are incompatible.
- *Increase of the braking distance of the train:* The braking distance of a train increases roughly in proportion to the square of speed. This fact, combined with the reduction of adhesion at high speeds, generates an extra increase in the power consumed, during the braking operation.
- *Requirements for high tractive power:* The required tractive power increases in proportion to the third power of speed. The great nominal motor power required at high speeds, combined with the need of instant supply of great tractive powers to the network, makes the electrification of trains a necessary condition for the development of high speeds. At this point, it should be mentioned that, at speeds $V > 160$ km/h,

a discontinuity may be observed in the contact between pantograph/overhead power wires, resulting in problems concerning the electrification of trains.

- *Instability in straight paths:* The speed parameter involved in the expression and value of creep forces determines the lateral stability of the vehicles to a great extent. At lower speeds, the movement is stable. Over a specific speed, the movement becomes unstable, causing oscillations of high amplitude, contact of the wheel flanges with the rails and lateral forces that may cause lateral displacement of the track (Pyrgidis, 1990).
- *Special track geometry alignment requirements:* The following three individual problems are identified:
  - *High centrifugal forces in the horizontal alignment curves:* During the movement of a railway vehicle in curves, centrifugal forces develop in curves, the value of which increases in proportion to the square of speed. To reduce these forces, it is required to adopt great curvature radii on the horizontal alignment, and apply high cant in high-speed lines. An overly high cant value causes problems to the coexistence of fast and slow trains in the same network.
  - *High vertical accelerations in the curved segments of the vertical alignment:* During the passage of trains from the curved segments of the vertical alignment, the vertical accelerations increase in proportion to the square of speed.
  - *High impact of geometric track defects:* Numerous measurements taken in various networks have shown that the impact of dynamic loads on the track superstructure increases in proportion to the speed and is directly proportional to the ride quality of the track, that is, the geometric track defects.

    The geometric track defects comprise the main cause of additional dynamic stresses which are generated by the interaction track–rolling stock (Alias, 1977; Esveld, 2001).
- *Noise pollution of the surrounding environment:* For speeds of up to 300 km/h, the noise level increase is a function of the third power of speed, while for higher speeds, the acoustic annoyance increases in proportion to the sixth power of speed (La vie du rail, 1989).
- *Severity of accidents:* In case of collision between trains, or collision of trains with obstacles on the track, the material damage is definitely more severe and the likelihood of injuries is higher. The same also applies to the case of derailment.
- *Increase of vertical dynamic loads:* The increase of speed does not affect significantly the load change caused by the suspended masses of the vehicle (car body), since the vertical accelerations of the car body increase less quickly than the speed, and they may be restricted by reducing the natural frequency of the car body, or by ensuring a relatively good track quality (Alias, 1977).

  In contrast, the semi-suspended masses of the vehicle (bogies), and particularly, the unsuspended masses (wheelsets) change significantly with the increase of speed, and increase the total value of the vertical dynamic load (Pyrgidis, 1990).
- *Passage over switches and crossings:* A good operation of high-speed network requires the passage of trains over switches and crossings with speeds higher than those applied in conventional networks. This requirement automatically generates new requirements in terms of the design and construction of switches and crossings.
- *Reduction of the dynamic comfort of passengers:* The increase of speeds automatically implies the increase of vertical and lateral accelerations of the car body, which have direct impact on the dynamic comfort of passengers.
- *Intensity of the aerodynamic effects and their impact on the 'open' track and platforms:* In 'open' track sections, the following may occur:

- During the passage of a train at high speed, the pressure along the lateral surface of the train as well as in the area adjacent to it changes and vibrations may affect the residential window panes located near the railway track.
- During the crossing of trains travelling in opposite directions at high speed, the pressure distribution along the trains affects their dynamic behaviour.

  On platforms, the air flow field that is generated intensifies with speed and may cause the following:
- Loss of balance, difficulty in walking and passengers or staff on the platforms near the tracks to be pushed violently.
- Ballast turbulence with the risk of injuring people on the platforms and causing damage to the rolling stock.

12.3 SPECIFICATIONS AND TECHNICAL SOLUTIONS FOR THE ACHIEVEMENT OF HIGH SPEEDS

12.3.1 Track geometry alignment characteristics

The application of high speeds automatically implies reviewing the geometry of the track alignment, and necessarily sets new values concerning the cant of track, the longitudinal slopes, the radii of curvature of the horizontal and vertical alignment, the distance between track centres and the tolerance limits for the geometric track defects.

12.3.1.1 Selection of horizontal alignment radii

Given that using conventional bogies it is not possible to ensure simultaneously high speeds in straight paths, and good inscription of wheelsets in curves, the horizontal alignment of the track layout should be characterised by the highest possible rate of straight paths, and by curved sections with the highest possible radii of curvature.

The selection of the horizontal alignment radii should be made based on two criteria (Pyrgidis, 2003):

1. The ‘physical’ behaviour of the vehicle, by applying the mathematical equations

$$R_{cmin} = \frac{11.8V_{max}^2}{U_{max} + I_{max}} \tag{12.2}$$

$$R_{cmin} = \frac{11.8(V_{max}^2 - V_{min}^2)}{E_{cmax} + I_{max}} \tag{12.3}$$

where

V_{max}, V_{min}: maximum speeds, respectively, of the fastest and slowest trains that operate in the track (km/h)

U_{max}: maximum permissible track cant (mm)

I_{max}: maximum permissible track deficiency (mm)

$$I_{max} = \frac{2e_o}{g}\gamma_{ncmax} \tag{12.4}$$

E_{cmax}: maximum permissible track excess (mm)
R_{cmin}: minimum curvature radius in horizontal alignment (m)
γ_{ncmax}: maximum permissible residual lateral acceleration (m/s^2 or g)
$2e_o$: theoretical distance between the running surfaces of the right and the left wheel when centred ≈ distance between the vertical axis of symmetry of the two rails = 1,500 mm (normal gauge track)

From the results of relations (12.2) and (12.3), the higher value of R_{cmin} is retained.

In case that, all the trains in the network run at the same speed, only relation (12.2) applies (it is taken $160 \leq U_{max} \leq 200$ mm).

The above relations take into account

a. The passage speed of the fastest and slowest train operating on the track
b. The categories of the operating trains (passenger, freight, etc.) and their rate of proportion
c. The behaviour of the human body in lateral accelerations and ensure, for the desired speeds, the selection of values for the horizontal alignment radii that meet the lateral dynamic comfort of passengers (physical behaviour of the vehicle)

   By contrast, they ignore the constructional characteristics of the rolling stock that will operate on track to a great extent.

2. The 'geometric' behaviour of the vehicle

   Regardless of the results arising from the analytical calculation, it should be examined, mainly for speeds $V \geq 250$ km/h, whether the constructional characteristics of the bogies of the rolling stock which are going to circulate on the track allow, for the specific radii of curvature, the proper inscription of the vehicle bogies (Pyrgidis, 2003).

   Specifically, if the bogies are characterised as too stiff on the level of the primary suspension (e.g., longitudinal and transversal stiffness K_x, $K_y > 10^7$ N/m, see Chapter 3), then the value of the horizontal alignment radius, which was calculated analytically, should increase by a rate.

   Table 12.1 displays indicatively for

a. $V_{max} = 300$ km/h, $U_{max} = 180$ mm, $\gamma_{ncmax} = 0.2/0.5/0.7$ m/s^2, $K_y = 10^7$ N/m.
b. And for various values of the longitudinal stiffness K_x of the primary suspension of vehicles.
   i. The theoretically minimum horizontal alignment radius R_{co}, which ensures the inscription of bogies without wheel–flange contact, and without wheel slippage. This value arises from the application of simulation models that describe the lateral behaviour of railway vehicles (see assumptions in Chapter 3) (Joly and Pyrgidis, 1990; Pyrgidis, 1990; Pyrgidis, 2003).
   ii. The minimum horizontal alignment radius R_{cmin} that arises from the application of mathematical equation (12.2) and which does not take into account the stiffness of the primary suspension.
   iii. The % rate by which the value of the above radius R_{cmin} should be increased, for the value R_{co} to arise, emerging from the application of simulation models.

At this point, it should be highlighted that the final rate of increment depends on the landscape and on any impact of the track layout on the surroundings built environment.

12.3.1.2 Distance between track centres

In high-speed networks, regardless of the traffic load, a double track is exclusively used for safety reasons. The track gauge is normal or broad.

Table 12.1 Selection of horizontal alignment radii on high-speed tracks

	Horizontal alignment radius R_{co} (m) as it derives from simulation models			*Horizontal alignment radius R_{cmin} as it derives from the analytical relation (12.2) and the rate of its required increment*		
$V_{max} = 300$ km/h $U_{max} = 180$ mm	$\gamma_{ncmax} = 0.2$ m/s^2	$\gamma_{ncmax} = 0.5$ m/s^2	$\gamma_{ncmax} = 0.7$ m/s^2	$\gamma_{ncmax} = 0.2$ m/s^2 $R_c = 5{,}042$ m	$\gamma_{ncmax} = 0.5$ m/s^2 $R_c = 4{,}140$ m	$\gamma_{ncmax} = 0.7$ m/s^2 $R_c = 3{,}699$ m
$K_x = 3.5 \times 10^7$ N/m	6,310	6,850	7,120	25.1%	65.4%	92.5%
$K_x = 1.5 \times 10^7$ N/m	5,770	6,130	6,310	14.4%	48.0%	70.6%
$K_x = 8 \times 10^6$ N/m	4,870	5,140	5,320	0%	24.1%	43.8%
$K_x = 5 \times 10^6$ N/m	3,970	4,150	4,330	0%	0%	17.0%
$K_x = 2 \times 10^6$ N/m	1,990	2,080	2,170	0%	0%	0%

In networks of track design speeds at $V_d = 200$ km/h, the minimum distance between track centres is set at $\Delta = 4.20$ m.

In networks of track design speeds at $V_d = 300$ km/h, the minimum distance between track centres is set at $\Delta = 5.00$ m.

The increase of the distance between track centres at these speeds is imposed due to the intensification of the aerodynamic effects in the event of crossing of trains travelling in opposite directions.

12.3.1.3 Longitudinal slopes

In the case of operation of exclusively passenger trains, due to lower axle load, lighter trains and electrification, high longitudinal slopes, in the region of $i = 3\%–4\%$, may be applied. The adoption of high longitudinal slopes limits the extent of the civil engineering structures.

12.3.2 Track superstructure components

The better ride quality of the track superstructure allows the reduction of the vertical dynamic stresses, ensuring a smoother rolling of the trains and better track robustness. The systematic operation of high-speed trains, as well as the numerous tests, confirmed the following:

- Use, throughout the network, of exclusively continuous welded rails (CWR)
- Fixing of sleepers/rails, assisted by exclusively double elastic fastenings and the use of heavy rails of UIC 60 type (60 kg/m)
- Use of concrete sleepers (monoblock or twin-block sleepers)
- Adoption of ballast width of 30–35 cm with high-hardness filling materials or, alternatively, the adoption of slab track
- Homogenisation of track stiffness characteristics

With regard to crossings, the use of swing nose crossings with a movable point frog (Figure 12.2) is required. This technology offers the possibility of high-speed passage, both at the main branch and at the diverging branch of a junction.

12.3.3 Civil engineering structures

12.3.3.1 Tunnel traffic

The reduction of the aerodynamic impacts that emerge in a tunnel is achieved by reducing the ratio

$$\frac{S_c}{S_u}$$

where
S_c the affected cross-sectional surface of the train
S_u the area of the useful cross section of the tunnel (Figure 12.3)

This ratio is called 'blockage ratio drag coefficient' of the train in the tunnel, and (Profillidis, 2014)

- For single-track tunnels it is considered equal to 0.30–0.50
- For double-track tunnels it is considered equal to 0.14

Figure 12.2 Movable point frog in a junction area. (From Voestalpine, 2015.)

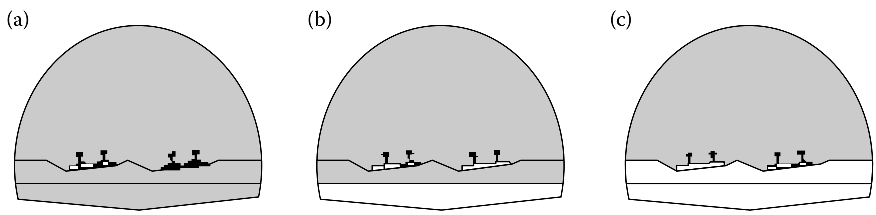


Figure 12.3 Cross sections of railway tunnels: (a) excavated, (b) effective, and (c) useful.

Table 12.2 displays the required area of the useful cross section S_u in relation to passage speed V_p in a double-track tunnel, for the cross section of the frontal surface of the vehicle, of approximately $S_c = 10$ m².

The increase of the useful cross section reduces all the aerodynamic impacts mentioned in Section 12.2.

- *Operation exclusively of trains with airtight car-body structure:* The use of special trains reduces the annoyance of passengers from the fluctuation in pressures exerted frontally and laterally on the train.
- *Adopting a twin-bore tunnel instead of a single-bore double-track tunnel:* With twin tunnels, the crossing of trains travelling in opposite directions is prevented, resulting in the reduction of waves from outer pressures that may cause problems to the passengers, the rolling stock and the freight.
- *Specially shaping the tunnel inlets (Figure 12.4):* With this shaping, the 'tunnel boom' effect is reduced. The area of the lateral cross section of an entrance is usually 1.4 times greater than that of the lateral cross section of the tunnel.

Table 12.2 Required useful cross section S_u of a double-track tunnel for high speeds

V_{pmax} (km/h)	160	200	240	300
S_u (m²)	40	55	71	100

Figure 12.4 High-speed tunnel entrance, Reisberg Tunnel, Germany. (Photo: A. Klonos.)

12.3.3.2 Passage under bridges

The operation of trains at very high speeds requires special handling, with regard to the selection of the height clearance h of the rolling surface of the rails from the lower surface of the bridge carrier (Figure 12.5).

This distance, in an electrified railway line, also depends, among other things, on the train-passage speed. The lifting of the contact wire, for speeds in the region of 230–300 km/h,

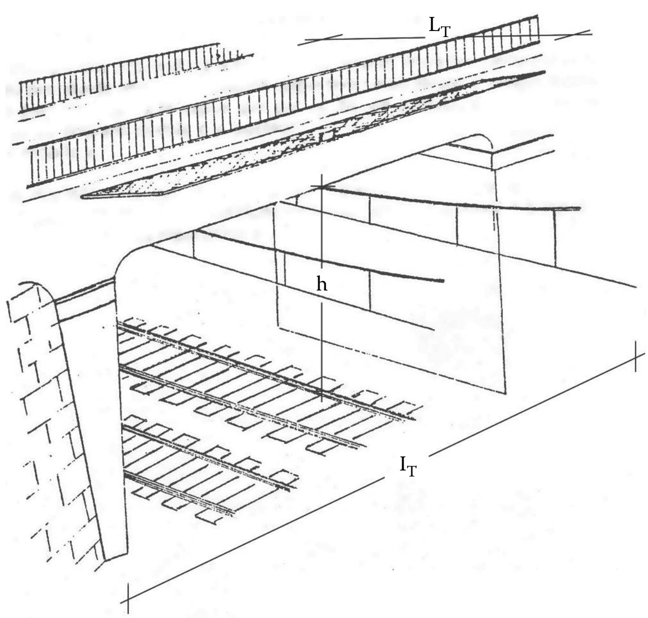


Figure 12.5 Passage of an electrified railway track under a road bridge – height clearance h, civil engineering structure width L_T, and civil engineering structure length l_T. (Figure based on SNCF 1984, *Ligne aérienne de traction électrique en courant alternatif monophasé 25 KV–50 Hz*, Internal document, Paris.)

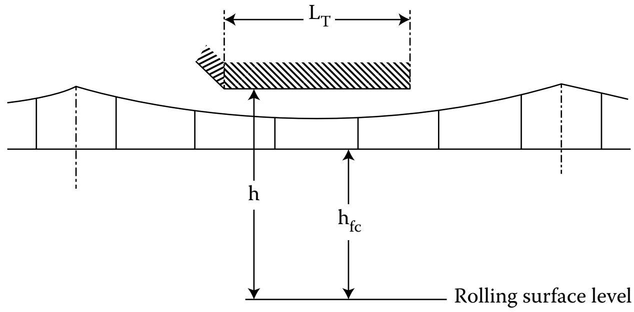


Figure 12.6 Civil engineering structures of small width ($L_T \leq 20$ m) – free passage of catenary. (Figure based on SNCF 1984, *Ligne aérienne de traction électrique en courant alternatif monophasé 25 KV–50 Hz*, Internal document, Paris.)

may take values in the region of 12 cm at the catenary support points, and slightly greater in the middle of the opening (SNCF, 1984; UIC 1986, 2003; Pyrgidis, 2004).

In the case that the civil engineering structure is of small width ($L_T \leq 20$ m), the installation of the catenary is performed by free passage, and the civil engineering structure is centred in the middle of the catenary opening for the best possible utilisation of the available height (Figure 12.6).

Table 12.3 provides the values of height clearance h, for passage speeds $V_p = 250$ km/h, for various widths of civil engineering structures L_T and for various heights of the catenary contact wire h_{fc}, with $h_{fc} = 5.20–5.75$ m.

For the drafting of Table 12.3, it was assumed that

$$h_o = 0.10 \text{ m}, \quad C_{hmin} = 0.555 \text{ m}, \quad I_1 = 0.48 \text{ m}.$$

where

h_o: track uplifting following maintenance works
C_{hmin}: constructional height in the middle of the catenary opening
I_1: isolation distance of wire–grounded structures

As can be seen from Table 12.3, in cases of railway track passage under civil engineering structures with width $L_T > 35$ m, the required height clearance h increases steeply. Under these conditions, for reasons of economy of the construction, in cases of civil engineering structures of large width, another method may be adopted for their installation (Figure 12.7).

Table 12.3 Height clearances of an electrified railway line – civil engineering structure deck for various values of L_T and h_{fc} – passage speed $V_p = 250$ km/h – free catenary passage

	Height clearance h (m)							
h_{fc} (m) \ L_T (m)	15	20	25	30	35	40	50	60
5.20	6.38	6.42	6.47	6.53	6.59	6.68	6.87	7.10
5.35	6.53	6.57	6.62	6.68	6.74	6.83	7.02	7.25
5.50	6.68	6.72	6.77	6.82	6.89	6.98	7.17	7.40
5.75	6.93	6.97	7.02	7.08	7.14	7.23	7.42	7.65

Source: Adapted from Pyrgidis, C. 2004. *Technika Chronika*, 24(1–3), 75–80.

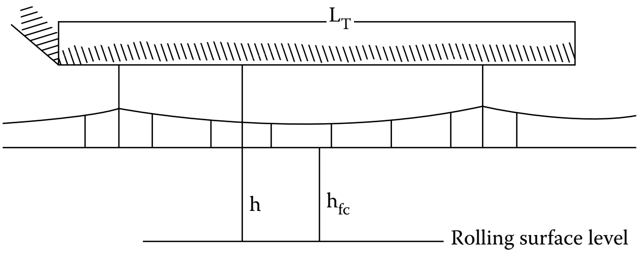


Figure 12.7 Civil engineering structures of large width – catenary fixing exclusively under the civil engineering structures. (Figure based on SNCF 1984, *Ligne aérienne de traction électrique en courant alternatif monophasé 25 KV–50 Hz*, Internal document, Paris.)

12.3.3.3 Track fencing

In high-speed networks, fencing on each side of the track is compulsory throughout the track. This aims (Figure 12.8)

- To reduce the likelihood of accidents which involve railway vehicles and pedestrians or animals (run-offs)
- To protect fauna and particularly larger mammals

12.3.3.4 Noise barriers

On the sides of the line, mainly in inhabited areas, noise-protection walls are constructed, specially designed in order to prevent noise transmission effects (Figure 12.9) (Rechtsanwalt et al., 2002).

The basic difference, compared with the noise barriers placed in highways, is due to the impact – upon train passage – of pressure–sub-pressure waves on the noise barriers, which should be taken seriously into account in dimensioning (Rechtsanwalt et al., 2002; Schweizer Norm. SN 671 250a, 2002).

Figure 12.8 Railway line with wire fencing on both sides, Rapsani, Greece. (Photo: A. Klonos.)

Figure 12.9 Concrete noise barriers, Aula Bridge, Germany. (Photo: A. Klonos.)

The size of this impact depends mainly on

- The square of speed of the passing train
- The aerodynamic shape of the train
- The shape–figure of the noise barrier
- The distance of the noise barrier from the track centre

A uniform load q is considered as a characteristic value of the impact of the pressure–sub-pressure wave, the value of which is given in Figure 12.10 in relation to the distance of the noise barrier from the track centre and for different speeds. For speed V = 300 km/h and lateral distance $a_d = 2.30$ m from the track centre, the maximum value of q is observed (q = 1.8 kN/m²) (Schweizer Norm. SN 671 250a, 2002).

Then the values of q that arise from the diagram of Figure 12.10 increase or decrease by a coefficient K or K′ depending on the type of the rolling stock and the dimensions of the noise barrier (Table 12.4).

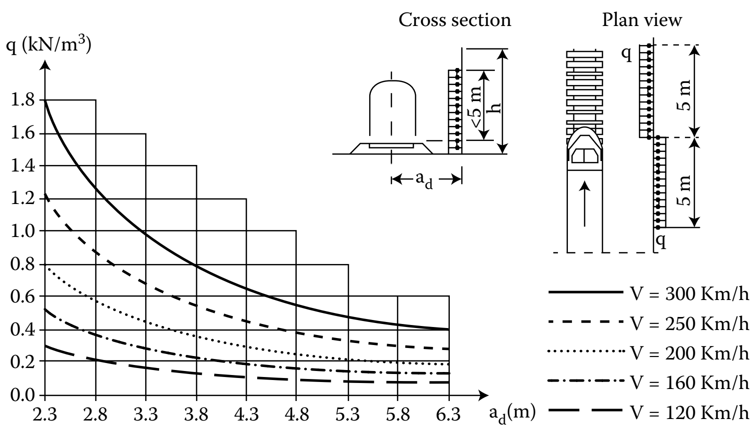


Figure 12.10 Change of the uniform load q in relation to the lateral distance a_d of the noise barrier from the track centre for different speeds. (Adapted from Schweizer Norm. SN 671 250a. 2002, *Schweizerischer Verband der Strassen – und Verkehrsfachleute* (VSS), May.)

Table 12.4 Values of the coefficients K and K′ for decrease and increase, respectively, of the aerodynamic load that apply to the noise barriers

Type of rolling stock	*K*
Trains with conventional shape	0.85
Trains with aerodynamic shape	0.60
Noise barrier dimensions	*K′*
Noise barrier height ≤ 1 m	1.30
Noise barrier length ≤ 2.50 m	1.30
Typical value	1.00

Source: Adapted from Schweizer Norm. SN 671 250a. 2002, *Schweizerischer Verband der Strassen – und Verkehrsfachleute (VSS)*, May 2002.

12.3.3.5 Handling aerodynamic effects in an 'open' track and on platforms

With respect to the 'open' track, the problems are handled through the aerodynamic design of trains, the increase of the distance between track centres and the placement of special protective walls and longitudinal noise barriers along the track.

With regard to the platforms, the problems are handled with the reduction of the train-passage speed in the station areas, the observance by the passengers and the staff on platforms, of a minimum permissible distance from the tracks and the installation of ventilation ducts on platforms (Baker, 2003; Baker and Sterling, 2003).

12.3.4 Track systems

In high-speed networks, the signalling system should provide for, at least in the sections where speeds of more than 220 km/h are developed, the ability to receive signals in the driver's cab. This signal is attained through special frequencies, which, through the track, transmits to special screens placed in the driver's cab, the current or the forthcoming indication to be observed. Thus, the train drivers receive, either on a permanent basis or instantly, information about the permissible speeds in various block sections.

A measure to reduce the likelihood of accidents in a high-speed network is the absence of railway level crossings.

12.3.5 Rolling stock

Interventions to the rolling stock concern mainly the aerodynamic design of vehicles, the design of bogies, the braking system and the dimensioning and construction of their frame.

12.3.5.1 Aerodynamic design of vehicles

12.3.5.1.1 Reduction of the coefficient K_l

The aerodynamic resistances W_α of the trains are given by the mathematical equation

$$W_\alpha = C_w V^2$$

where

$$C_w = K_1 S_c + K_2 L_{tr} p \tag{12.4}$$

K_1: a parameter that depends on the shape of the 'nose' and the 'tail' of the train
S_c: the lateral cross section of the affected surface of the train
L_{tr}: the train length
p: the perimeter that encloses the rolling stock laterally, up to rail level (rolling stock outline)
K_2: a parameter that depends on the construction of surface $p \cdot L_{tr}$

The reduction of coefficient K_1 allows the reduction of coefficient C_w, and as result, the reduction of the aerodynamic resistances.

The finite element method is the most suitable for the design of the aerodynamics of vehicles. It enables the application of air flow forces to every part of the vehicle, resulting in the shape of the vehicle that favours the development of high speeds, and minimises energy consumption.

12.3.5.1.2 Reduction of coefficient K_2

The diagram of Figure 12.1 lists all the constructional parameters of a vehicle that affect the aerodynamic resistances of a train, as well as the rate of contribution of each of them to the value of coefficient C_w. The shape of the 'nose' and the 'tail' of the train are directly related to the value of coefficient K_1, while all the other parameters concern coefficient K_2.

The use of trains with shared bogie vehicles is increasingly adopted. These fixed formation trains are equipped with special type bogies (Jakob-type bogies). In these trains, two vehicles are successively straddled on each 2-axle bogie, thereby reducing the number of bogies to $n_b/2 + 1$ (where n_b is the original number of bogies), and hence, the gap between the coupling of the vehicles, as well as the total weight of the train (Figure 12.11).

All the above conditions allow a significant reduction of coefficient K_2 and, hence, of the aerodynamic resistances.

12.3.5.2 Design of bogies

Vehicles intended to move at very high speeds are equipped, in their vast majority, with conventional bogies (see Section 3.2.1). Given that with this type of bogies it is not possible

Figure 12.11 Articulated train formation with shared bogie vehicles, TGV-ATLANTIQUE, bogie Y-237. (Adapted from Fabbro, SNCF Médiathèque 1989.)

to attain simultaneously high speeds in straight paths and good inscription of wheelsets in curves, a large percentage of the network's length is constructed in straight path, and the constructional parameters of the vehicle are optimised, in order to ensure high speeds in straight paths (see Section 3.3).

At the same time

- The profile of the wheel treads is fixed in frequent intervals (every 300,000 km), so that their initial conicity is maintained.
- The static axle load is reduced to 16–17 t.
- The unsuspended and semi-suspended masses (wheelsets, bogies) are reduced.

12.3.5.3 Braking system

Safe braking at high speeds is attained by combining various braking systems and devices, such as disc brakes made of specially processed steel, rheostatic braking, electromagnetic shoes and anti-lock braking system (wheel slide protection).

12.3.5.4 Vehicle design: Construction

From the conception of the idea of high-speed trains, the engineers' choices were guided by caution. The dimensioning of the rolling stock and the track is performed with high safety considerations. The durability of the framework of the vehicles is increased, and the bumpers of the leading vehicle are strengthened. At the same time, a series of auxiliary devices and automations ensure the smooth movement of the train in case of failure of specific functions, and warns the train drivers of potential problems.

12.3.5.5 Implementation cost

The infrastructure cost of a high-speed, double-track line varies from €10 to €40 M per track-km. It depends on the percentage of line length constructed with slab track, the length of civil engineering structures (tunnels, bridges) and the difficulty of the topographic relief.

12.4 HISTORICAL REVIEW AND CURRENT SITUATION OF HIGH-SPEED NETWORKS AND TRAINS

The application of high-speed networks began in Japan in 1964 with the operation of the Shinkansen train in the Tokyo–Osaka line (maximum running speeds $V_{max} = 210$ km/h, connection length S = 515 km).

In Europe, the operation of high-speed trains began in the early 1980s. The French railways were the first to operate high-speed trains in their network. Specifically, in the autumn of 1981, the TGV Paris Sud-Est train was routed in the Paris–Lyon new line, originally with a maximum running speed of $V_{max} = 260$ km/h which was increased (from 1983) to $V_{max} = 270$ km/h. It was followed, in the autumn of 1989, by the TGV Atlantique train, which was routed with $V_{max} = 300$ km/h on the homonymous corridor, serving areas in the western and southwestern France.

Today, the technical developments in the field of the rolling stock, as well as in the field of the track, allow a railway train to move in complete safety, in a straight path of good ride quality, at speeds of 350 km/h.

The operation of high-speed trains today occurs either in new lines with high-speed specifications or in upgraded existing tracks. In both cases, the trains use conventional or tilting technology.

Table 12.5 provides a summary of statistics on high-speed rail networks.

The data recorded and analysed in the following relate to the year 2014 for the length of the lines and 2015 for the speeds. The raw data were obtained both per country and per line, from various available sources and cross-checked. Afterwards they were further manipulated for the needs of this chapter.

The total length of lines where the developed running speeds V_{max} are greater than 250 km/h amounts to 27,441 km in the world. China has the longest lines (S = 16,293 km) holding a percentage of 59.4%, followed by Spain (S = 2,427 km, 8.9%), Japan (S = 2,346 km, 8.5%) and France (S = 1,905 km, 7.0%).

Moreover, China has the greatest length of lines where the developed running speeds V_{max} are greater than 200 km/h (S = 18,912 km, share 51.8%) in the world.

Table 12.6 provides the high-speed railway lines per country, their length, the year in which they commenced operation and the maximum running speed that can be developed (UIC, 2014b; Wikipedia, 2015b).

Table 12.7 displays the basic constructional and operational characteristics of high-speed trains (without tilting car body) that operate currently in new and upgraded high-speed tracks worldwide.

Table 12.8 displays the basic constructional and operational characteristics of high-speed trains (without tilting car body) which are going to operate in the coming years.

Table 12.5 Summarised data for high-speed railway networks

	Country	*Length (km)* $V_{max} \geq 250$ *km/h* ($V_{max} \geq 200$ *km/h)* *(2014 data)*	*Percentage over total length (%)* $V_{max} \geq 250$ *km/h* ($V_{max} \geq 200$ *km/h)* *(2014 data)*	V_{max} *(km/h)*	V_{ar} *(km/h)/* L_{st} *(km)* *(2015 data)*
1	China	16,293 (18,912)	59.4 (51.8)	300	283.4/382.6
2	Spain	2,427 (2,432)	8.9 (6.7)	310	259.6/242.3
3	Japan	2,346 (2,616)	8.5 (7.2)	320	267.4/294.1
4	France	1,906 (1,906)	7.0 (5.2)	320	271.8/167.6
5	Italy	959 (959)	3.5 (2,6)	300	232.2/205.1
6	Germany	807 (2,509)	2.8 (6.9)	300	245.7/143.3
7	Turkey	745 (745)	2.7 (2.0)	250	217.4/221.0
8	Russia	630 (1,270)	2.5 (3.5)	250	194.5/201.0
9	South Korea	536 (766)	2.0 (2.1)	300 (305)	210.5/133.3
10	Taiwan	339 (345)	1.2 (0.9)	300	256.4/179.5
11	Belgium	214 (214)	0.6 (0.6)	300	239.2/291.0
12	Netherlands	125 (125)	0.5 (0.3)	300	178.1/95.0
13	United Kingdom	114 (1,391)	0.4 (3.8)	300	177.9/186.8
14	Sweden	0 (957)	0.0 (2.6)	200	168.1/179.3
15	United States	0 (730)	0.0 (2.0)	225	173.8/110.1
16	Uzbekistan	0 (344)	0.0 (1.0)	200 (210)	161.3/344.0
17	Austria	0 (292)	0.0 (0.8)	230 (250)	156.6/133.0
	Total	27,441 (36,513)	100.0 (100.0)		

Source: Adapted from Hartill, J. 2015, *Railway Gazette International*, July, 44–48; UIC. 2014b, High speed lines in the world, *International Union of Railways* [online], available from: http://www.uic.org/IMG/pdf/20140901_high_speed_lines_in_the_world.pdf (accessed 30 April 2015); Wikipedia. 2015b, *Schnellfahrstrecke*, online, available from: http://de.wikipedia.org/wiki/Schnellfahrstrecke (accessed 30 April 2015).

Table 12.6 High-speed railway lines per country

Line	*Length (km)*	*Starting year of operation*	V_{max} *(km/h)*
Japan			
Tokyo–Shin Osaka (Tokaido line)	515	1964	285
Shin Osaka–Hakata (Sanyo line)	554	1972/1975 (in stages)	300
Tokyo–Shin Aomori (Tohoku line)	675	1982/1910 (in stages)	320–260
Omiya–Niigata (Joetsu line)	270	1982	240
Takasaki–Nagano (Hokoriku line)	117	1997	260
Nagano—Kanazawa (Hokoriku line)	228	2014	260
Hakata–Kagoshima-Chuo (Kyushu line)	257	2004/2011	260
Total	2,616		
France			
Paris–Lyon (TGV Paris–Sud-Est)	409	1981/1983 (in stages)	300
Paris–Le Mans/Tours (TGV Atlantique)	284	1989/1990 (in stages)	300
Lyon–Valence and Lyon detour (TGV Rhône-Alpes)	115	1992/1994 (in stages)	300
Interconnection TGV Nord–TGV Sud-Est	57	1994/1996 (in stages)	300
Paris–Lille–Calais (TGV Nord Europe)	333	1993	300
Valence–Marseille/Nimes (TGV Mediterranée)	243	2001	320
Vaires-sur-Marne–Baudecourt (TGV Est)	300	2007	320
Figueres–Perpignan (French section)	25	2011	300
Villers les Pots–Petit Croix (TGV Rhin–Rhone–part of the eastern branch)	140	2011	320
Total	1,906		
Spain			
Madrid–Seville	472	1992	300
Madrid–Barcelona	621	2003/2008 (in stages)	310 (350)
Cordoba–Malaga	155	2006/2007 (in stages)	300 (350)
Madrid–Valladolid	179	2007	300
Madrid detour	5	2009	200
(Madrid)–Valencia	363	2010	300 (350)
Motilla del Palancar–Albacete	63	2010	300 (350)
Ourense–A Corouna (1668-mm track gauge to be converted to normal gauge)	152	2011	300 (350)
Albacete–Alicante	171	2013	300 (350)
(Madrid)–Toledo	21	2005	250
Barcelona–Figueres	131	2013	300
Figueres–Borders	20	2010	300
Saragossa–Tardienta (Huesca)	79	2003	200
Total	2,432		
Germany			
Hanover–Würzburg	327	1988/1991 (in stages)	280
Mannheim–Stuttgard	99	1991	250

(*Continued*)

Table 12.6 (Continued) High-speed railway lines per country

Line	*Length (km)*	*Starting year of operation*	V_{max} *(km/h)*
Augsburg–Munich–Olching (Munich–Augsburg)	43	1977–2011	200–230
Hamm–Bielefeld (Railway Hamm–Minden)	67	1980	200
Augsburg–Donauwörth (Nuremberg–Augsburg)	36	1981	200
Hanover–Würzburg	327	1988/1991(in stages)	280 (250 in tunnels)
Cologne–Duisburg	64	1991	200
Mannheim–Stuttgart	99	1991	280 (250)
Dinkelscherben–Augsburg	20	1992	200
Hanau–Gelnhausen (Kinzigtal Bahn)	16	1993	200
Berlin–Hanover	258	1998	160–250
Koln–Frankfurt (Cologne–Rhine/Main)	177	2002/2004 (in stages)	300
Munster–Bremen–Hamburg (Wanne-Eickel–Hamburg Railway)	288	1982/1991	200
Mannheim–Frankfurt	78	1991	200
Leipzig–Riesa (Leipzig–Dresden)	66	2002	200
Nuremberg–Ingolstadt (Nuremberg–Munich)	89	2006	300
Munich–Petershausen (Nuremberg–Munich)	29	2006	300
Berlin–Halle/Leipzig	187	2006	200
Erfurt–Leipzig/Halle	123	2015	300
Koln–Duren (Koln–Aachen)	42	2003	250
Rastatt South Offenburg (Karlsruhe–Basel)–	44	2004	250
Hanover–Hamburg	170	1987	200
Hamburg–Berlin	286	2004	230
Total	2,509		
Italy			
Rome–Florence	254	1978/1992 (in stages)	250
Rome–Naples	205	2006–2009 (in stages)	300
Turin–Milan	125	2006/2009 (in stages)	300
Padova–Venice (Mestre)	25	2007	300
Bologna–Florence	79	2009	300
Milan–Bologna	215	2008/2009 (in stages)	300
Milan–Treviglio	27	2007	300
Naples–Salerno	29	2008–2009	250
Total	959		
South Korea			
Gyeongbu HSR corridor (phase 1 and 2)	412	2004/2014 (in stages)	305 (350)
Osong–Gwangju–Songjeong (Honam line)	174	2014	300
Iksan–Yeosu Expo (Jeolla line)	180	211	230
Total	766		
United Kingdom			
Channel Tunnel Rail Link (sections 1 + 2)	114	2003/2007	300
London–Newcastle–Edinburgh (East Coast Main Line)	632	2000	200

(*Continued*)

Table 12.6 (Continued) High-speed railway lines per country

Line	*Length (km)*	*Starting year of operation*	V_{max} *(km/h)*
London Euston–Rugby–Edinburgh/Glasgow (West Coast Main line)	645	2004	200
Total	1,391		
Taiwan			
Taipei–Kaohsiung	345 (339)	2007	293 (300)
Total	345		
China			
Beijing–Shanghai (via Xuzhou–Nanjing)	1,318	2008–2011	350
Urumqi–Lanzhou	1,776	2014	250
Baoli–Xian	148	2013	300
Xian–Zhengzhou	455	2010	350
	2,379		
Guiyang–Liuzhou–Guangzhou	856	2014	300
Jiangyou–Chengdu–Leshan	314	2014	200
	1,170		
Shanghai–Hangzhou	150	2010	350
Hangzhou–Changsha	933	2014	350
Changsha–Huaihua	416	2014	300
	1,499		
Beijing–Wuhan	1,119	2012	350
Wuhan–Guangzhou	968	2009	350
Guangzhou–Shenzhen–Hong Kong Mainland	106	2011	250
	2,193		
Changchun–Jilin	111	2010	250
Haerbin–Dalian	904	2012	350
Panjin–Yinkou	89	2013	350
	1,104		
Tianjin–Qinhuangdao	261	2013	350
Qinhuangdao–Shenyang	404	2003	250
	665		
Nanjing–Hefei	166	2008	250
Hefei–Wuhan	351	2009	250
Wuhan–Yichang	293	2012	250
Yichang–Chengdu	921	2009–2013	200
	1,731		
Nanjing–Hangzhou	251	2013	350
Hangzhou–Ningbo	152	2013	350
Ningbo–Wenzhou–Fuzhou–Xiamen	841	2009–2010	250
Xiamen–Shenzhen	502	2013	250
	1,746		
Hengyang–Liuzhou	498	2013	200
Liuzhou–Nanning	223	2013	250

(*Continued*)

Table 12.6 (Continued) High-speed railway lines per country

Line	*Length (km)*	*Starting year of operation*	*V_{max} (km/h)*
	721		
Longyan–Xiamen	171	2012	200
Ganzhou–Longyan	114	2012	200
	285		
Jinan–Qingdao	364	2008	250
Shijiazhuang–Taiyuan	190	2009	250
Qingdao–Rongcheng	299	2014	250
	853		
Qinzhou–Beihai	100	2013	250
Nanning–Qinzhou–Fangchenggang	162	2013	250
	262		
Nanchang–Jiujiang	131	2010	250
Nanchang (Xiangtang)–Putian	635	2013	200
	766		
Guangzhou–Zhuhai (main line)	117	2012	200
Guangzhou–Zhuhai (Xinhui branch)	27	2011	200
	142		
Xian–Taiyuan	570	2014	250
Hefei–Bengbu	131	2012	300
Nanning–Guangzhou PDL	577	2014	250
Chengdu–Dujiangyan	86	2010–2014	200–220
Haikou–Sanya (Hainan Eastern Ring Railway)	308	2010	250
Wuhan Metropolitan	253	2013–2014	250
Zhengzhou–Kaifeng (Central Plain Metropolitan Intercity Rail)	50	2014	200
Maoming–Zhanjiang	103	2013	250
Total	18,912		
United States			
Boston–Washington (Northeast Corridor)	730 (161)	2000	215 (240)
Total	730		
Turkey			
Ankara–Istanbul	533	2009–2014	250
(Ankara)–Polatli–Konya	212	2011	250
Total	745		
Sweden (Trafikverket, 2014)			
Katrineholm–Malmö (South main line Stockholm–Malmö)	483	1992	200
Stockholm–Gothenburg (West main line)	455	1990	200
X3 Stockholm Central Station–Arlanda Airport (Arlanda line)	19	1999	200
Total	957		

(*Continued*)

Table 12.6 (Continued) High-speed railway lines per country

Line	*Length (km)*	*Starting year of operation*	V_{max} *(km/h)*
Netherlands			
HSL-Zuid Schiphol–Dutch border	125	2009	300
Total	125		
Belgium			
Brussels–French border (HSL-1)	71	1997	300
Leuven (Brussels)–Liege(HSL-2)	61	2002	300
Liege–German border (HSL-3)	42	2009	260
Antwerp (Brussels)–Dutch border (HSL-4)	40	2009	300
Total	214		
Russia			
Moscow–St Petersburg	650	2009	250
Finnish border–St Petersburg	160	2010	200
Moscow–Nizhny Novgorod	460	2010	250 (at small parts)
Total	1,270		
Uzbekistan			
Tashkent–Samarkand	344	2011	210 (250)
Total	344		
Austria			
Western Railway (Vienna–St Polten–Linz–Salzburg)	43	250–230	2012
Vienna–St Polten	133	200–230	2001–2014
St Polten–Linz	25	200	1993
Linz–Wels	51	200–230	2012
Wels–Punchheim			
New Unterinntalbahn (Kundl–Baumkirchen)	40	2012	220
Total	292		

12.5 INTEROPERABILITY ISSUES

The term ‘railway interoperability’ implies the capacity of the trans-European railway system to allow safe and continuous circulation of trains among its various segments, achieving the required performance in specific lines. This capacity is ensured by a set of regulatory, technical and operational requirements which must be satisfied. These requirements were set by Directive 96/48 which also established the above definition.

The railway interoperability concerns two different cases of railway operation:

- *High-speed networks* and specifically the operation of trains in the categories of tracks (I, II, III) as they were defined in Section 12.1.
- *Conventional-speed networks:* The conventional-speed networks include new tracks which are designed for speeds less than 200 km/h or for existing tracks which are upgraded. However, the track design speeds remain less than 200 km/h.

Table 12.7 High-speed conventional technology trains worldwide (indicative table)

Name and type of train	*Operator*	*Maximum running speed (rolling stock design speed) (km/h)*	*Total nominal motor's power (kW)*	*Transport capacity (passengers)*	*Train length (m)*	*First operation*	*Figure*
AVE Class 100 TGV	RENFE	300 (300)	8,800	329	200.2	1992	
KTX-Sancheon Hyundai-Rotem	Korail	300 (305)	8,800	363	201	2010	
Eurostar TGV	Eurostar	300 (300)	12,200	770	393.7	1993	
ICE 3 Class 407 Velaro	DB	320 (320)		406		2011	

(Continued)

Table 12.7 (Continued) High-speed conventional technology trains worldwide (indicative table)

Name and type of train	*Operator*	*Maximum running speed (rolling stock design speed) (km/h)*	*Total nominal motor's power (kW)*	*Transport capacity (passengers)*	*Train length (m)*	*First operation*	*Figure*
THSR 700T Shinkansen	THSR	300 (300)	10,260	989	304	2007	
BR Class 395 A-train	Southeastern	225 (225)	3,990	340	121.3	2009	
Series 800 Shinkansen	JR Kyushu	260 (285)	6,600	392	154.7	2004	
Series E1 (double decker) Shinkansen	JR EAST	240	9,840	1,235	302.1	1994	

(*Continued*)

Table 12.7 (Continued) High-speed conventional technology trains worldwide (indicative table)

Name and type of train	*Operator*	*Maximum running speed (rolling stock design speed) (km/h)*	*Total nominal motor's power (kW)*	*Transport capacity (passengers)*	*Train length (m)*	*First operation*	*Figure*
TGV Atlantique	SNCF	300 (300)	8,800	485	237.5	1989	
TGV Duplex (double decker)	SNCF	320 (320)	8,800	512	200	1995	
Thalys PBKA TGV	Thalys	300 (320)	8,800	377	200	1997	
TCDD HT 65000 (talgo)	TCDD	250 (250)	4,800	419	158.5	2009	

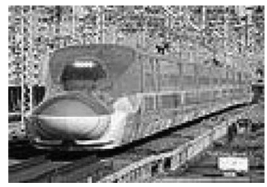
(*Continued*)

Table 12.7 *(Continued)* High-speed conventional technology trains worldwide (indicative table)

Name and type of train	*Operator*	*Maximum running speed (rolling stock design speed) (km/h)*	*Total nominal motor's power (kW)*	*Transport capacity (passengers)*	*Train length (m)*	*First operation*	*Figure*
Railjet Taurus	OBB	230 (230)	6,400	316	294.78	2008	
CRH380D & DL Zefiro	China Railway Corporation	300 (380)	10,000	664	215.3	2012	
ETR 575 AGV	NTV.Italo	300 (360)	6,080	245	132.1	2011	
Afrosiyob Talgo 250	Uzbekistan Railways	220 (250)	5,056	257	157	2011	

Source: Adapted from Wikipedia 2015a, *List of high-speed trains*, online, available from: en.wikipedia.org/wiki/List_of_high-speed_trains (accessed 30 April 2015).

Table 12.8 High-speed conventional technology trains that are going to get into circulation in the coming years

Name and type of train	*Operator*	*Train design speed*	*Starting operation year*	*Figure*
ETR 1000 Zefiro	Trenitalia	400 km/h	2015	
KTX-III Hyundai-Rotem	Korail	370 km/h	2015	
Series H5 Shinkansen	JR Hokkaido	320 km/h	2016	
Series W7 Shinkansen	JR West	275 km/h	2015	
Series E7 Shinkansen	JR East	275 km/h	March 2014	
Eurostar e320 Velaro	Eurostar	320 km/h	2015	

In order for the trans-European network to be implemented, the railway interoperability required the high-speed railway system to be analysed in subsystems. These subsystems are described in detail in Annex II of Directive 96/48 and they are the following:

- Infrastructure
- Rolling stock
- Energy
- Control – command and signalling

- Operation and traffic management
- Maintenance
- Telematic applications for passenger and freight services

The issuance of Directive 96/48 marked the beginning of the development and drafting of the TSI for each of the above subsystems, and which comprise the essential elements for the achievement of interoperability.

The TSI has been completed for all the subsystems, with effective date from January 2003. However, issues of TSI have not taken yet their final form, as minor amendments are made in various sections.

At this point, it should be noted that the competent body for the final configuration of the TSI is the European Railway Agency (ERA).

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Chapter 13

Tilting trains

13.1 DEFINITION AND FUNCTION PRINCIPLE OF TILTING TECHNOLOGY

Tilting trains (Pendolino, trains à caisse inclinable, Neigezug) are conventional railway trains equipped with special technology that allows the car body of all vehicles, when it negotiates a horizontal curve, to rotate with respect to its longitudinal axis by an angle φ_t (tilting angle). As a result, the total rotation angle of the car body with respect to the horizontal level is equal to the sum of the track's cant angle δ_p plus the tilting angle φ_t (Figures 13.1 through 13.3).

As analysed in Figure 13.1, tilting technology further increases the lateral component of the vehicle's weight B_{ty} resulting in a decrease of the effect of the centrifugal force F_{nc} on the passengers, thereby improving their lateral dynamic comfort.

a. Conventional railway vehicle ($F_{nc} = F_{cfy} - B_{ty} = F_{cf} \cos \delta_p - B_t \sin \delta_p$)
b. Tilting railway vehicle ($F'_{nc} = F'_{cfy} - B'_{ty} = F_{cf} \cos(\delta_p + \varphi_t) - B_t \sin(\delta_p + \varphi_t)$), $F'_{nc} < F_{nc}$

where

F_{nc}, F'_{nc}: residual centrifugal force
F_{cf}: centrifugal force
B_t: vehicle's weight
B_{ty}, B'_{ty}: lateral component of the vehicle's weight
B_{tz}, B'_{tz}: vertical component of the vehicle's weight

On the basis of the above, tilting technology may potentially result in the following two alternative improvements regarding the level of service provided to passengers:

- It can ensure lower residual centrifugal force $F'_{nc}(F'_{nc} < F_{nc})$ on the car body's level in curved segments of the track for the same passage speed V_p compared with conventional trains. As a result, it can also ensure a smaller lateral residual acceleration $\gamma'_{nc}(\gamma'_{nc} < \gamma_{nc})$ and thus a higher level of lateral ride comfort for passengers. In this case, the residual centrifugal acceleration γ_{nc} at wheelsets' level and the forces exerted on the track (guidance forces F_{ij} in the case of flange contact, creep forces X_{ij}, T_{ij}) remain the same for both technologies (Table 13.1).
- It can enable an increase of the passage speed in curves, keeping the car-body level, the same value as that of the residual lateral acceleration γ_{nc} adopted for conventional trains (Table 13.2). This increase in speed does not affect the passenger's ride comfort;

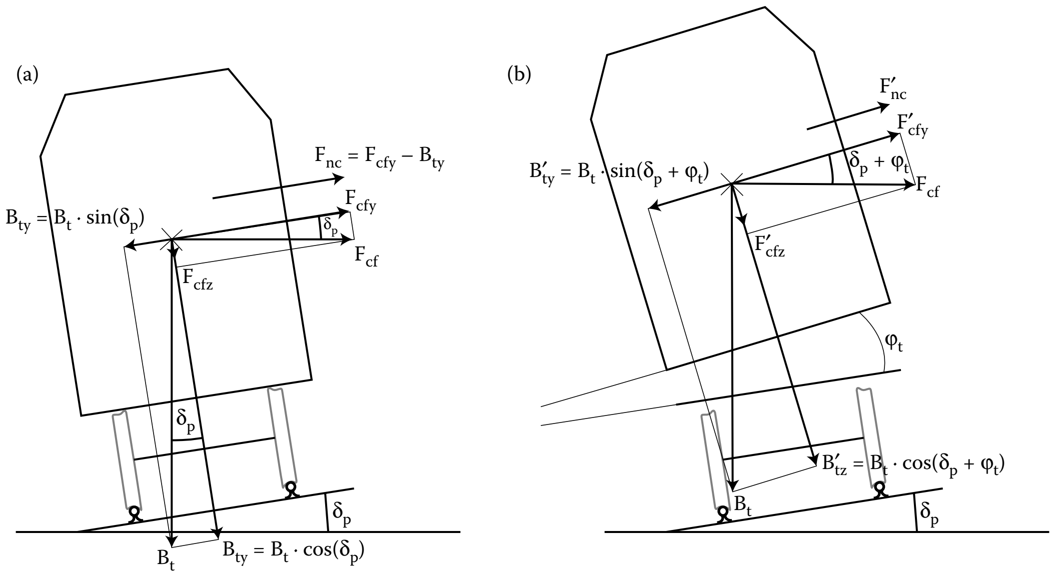

Figure 13.1 Railway vehicle motion in a curved section of the track. Track with cant. (a) Conventional railway vehicle ($F_{nc} = F_{cfy} - B_{ty} = F_{cf} \cdot \cos\delta_p - B_t \cdot \sin\delta_p$). (b) Tilting railway vehicle ($F'_{nc} = F'_{cfy} - B'_{ty} = F_{cf} \cdot \cos(\delta_p + \varphi_t) - B_t \cdot \sin(\delta_p + \varphi_t)$), $F'_{nc} < F_{nc}$ where F_{nc}, F'_{nc}: residual centrifugal force, F_{cf}: centrifugal force, B_t: vehicle's weight, B_{ty}, B'_{ty}: lateral component of the vehicle's weight, B_{tz}, B'_{tz}: vertical component of the vehicle's weight. (Adapted from Pyrgidis, C. and Demiridis, N. 2006, The effects of tilting trains on the track superstructure, *1st International Congress, 'Railway Conditioning and Monitoring' 2006*, IET, 29–30 November 2006, Birmingham, UK, Conference Proceedings, pp. 38–43; Pyrgidis, C. 2009, Tilting trains/conventional trains – Comparison of the lateral forces acting on the track, *Ingegneria Ferroviaria*, April 2009, Rome, No 4, pp. 361–371.)

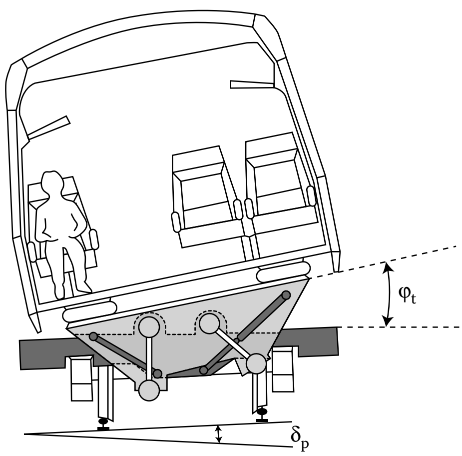

Figure 13.2 Tilting car-body railway vehicle negotiating a curved track section – Track with cant. (Adapted from ABB (no date), The fast-Train – technology for faster passenger services on existing railway lines, Leaflet, Manheim, Germany.)

Figure 13.3 Tilting car-body railway vehicle negotiating a curved track section, Silenen, Switzerland. (Photo: A. Klonos.)

Table 13.1 Tilting and conventional car-body trains running with the same speed – effects on horizontal curves

Car-body technology	*Passage speed*	*Lateral residual acceleration*		*Guidance forces/creep forces*
		Car-body level	*Bogies and wheelset level*	
Conventional car body	V_p	γ_{nc}	γ_{nc}	$F_{ij}/X_{ij}, T_{ij}$
Tilting car body	V_p	$\gamma'_{nc} < \gamma_{nc}$	γ_{nc}	$F_{ij}/X_{ij}, T_{ij}$

Source: Adapted from Pyrgidis, C. 2009, Tilting trains/conventional trains – Comparison of the lateral forces acting on the track, *Ingegneria Ferroviaria*, April 2009, Rome, No 4, pp. 361–371.

Table 13.2 Tilting trains running with higher speed than conventional car-body trains – effects on horizontal curves

Car-body technology	*Passage speed*	*Lateral residual acceleration*		*Guidance forces/creep forces*
		Car-body level	*Bogies and wheelset level*	
Conventional car body	V_p	γ_{nc}	γ_{nc}	$F_{ij}/X_{ij}, T_{ij}$
Tilting car body	$V'_p > V_p$	γ_{nc}	$\gamma'_{nc} > \gamma_{nc}$	$F'_{ij} > F_{ij}, X'_{ij}, T'_{ij} > X_{ij}, T_{ij}$

Source: Adapted from Pyrgidis, C. 2009, Tilting trains/conventional trains – Comparison of the lateral forces acting on the track, *Ingegneria Ferroviaria*, April 2009, Rome, No 4, pp. 361–371.

however, it has an impact on the track superstructure as it affects the geometrical positioning of the wheelsets on the track (bogies' curving ability) and, through this, an impact on the forces acting on the wheel–rail surface (Table 13.2).

On the basis of the above, tilting trains enable railway companies to reduce travel times while at the same time they can maintain their old infrastructure assets, provided that those are in good condition.

13.2 TILTING TECHNIQUES AND SYSTEMS

Depending on how the tilting of the car body is achieved, two tilting principles are distinguished, namely passive tilting and active tilting (Profillidis, 1998).

13.2.1 Passive tilting

Tilting is activated by the vehicle's inertia forces and not by some mechanism. The trains' vehicles are manufactured to have a lowered centre of gravity, in order for the centrifugal forces exerted on horizontal curves to force them to tilt. This principle was developed during the first attempts to apply tilt technology on trains (TurboTrain, USA, 1968) and it was applied successfully in the Talgo Spanish trains (Talgo tilting system). Passive tilting allows for the tilting angle φ_t between vehicle car body and bogie to have a value between 3° and 5°.

13.2.2 Active tilting

The main issue regarding active tilting lies with the initiation of the tilting at the right moment, that is, just before the vehicle enters the transition curves, and not with obtaining/maintaining the desired tilting angle. More specifically, the car body must be located at the desired position when the vehicle passes through the circular segment of the curve as well as when it returns to the straight segment of the track. In the opposite case, passengers will encounter a violent lateral acceleration which, although short term, will be annoying, if not unacceptable. The problem is even greater if the track alignment involves two successive reverse curves without an intermediate straight segment.

For this reason, the car body must have already obtained the desired tilting angle at the curve's transition curves. To achieve this, given that the time within which the tilting process must be completed is very limited (around 2 s), the car body's tilting must occur mechanically rather than by applying the principle of passive tilting.

Until today, two active tilting techniques have been developed, namely the European technique and the Japanese technique (Figure 13.4).

In the European technique, tilting is achieved using equipment that is mounted solely on the vehicle and is operated hydraulically or electrically. A gyroscope placed on the head of the train 'recognises' the cant of the outer rail in curves, and determines the tilting angle. At the same time, an accelerometer placed on the front bogie measures the lateral acceleration generated in curves and acts for the determination of the progressive tilting of the car body. These two equipment devices interact with a microcomputer unit that generates the necessary commands. This technique allows for a tilting angle between the car body and bogie of $\varphi_t = 8°$. It was adopted as a basic principle by various rolling stock manufacturers and was applied with variants (different technical versions). This technique is applied for the following tilting systems: Pendolino, Fiat SIG, Nuovo Pendolino, ASEA, Bombardier, CAF (SIBI), Adtranz and Siemens (Chiara et al., 2008).

In the Japanese technique, tilting is achieved using equipment that is placed both on the track and on the vehicle (Figures 13.4 and 13.5). The technique combines passive tilting, that is, the natural pendulum function using the centrifugal force, with a pneumatic pressure (air) mechanical system. The identification of the track is achieved via an electromagnetic system that detects the curve. The geometric characteristics of the track are recorded for the entire length of the route in an electronic format using a computer that is mounted on the vehicle. The position of the train on the track is calculated every moment in relation

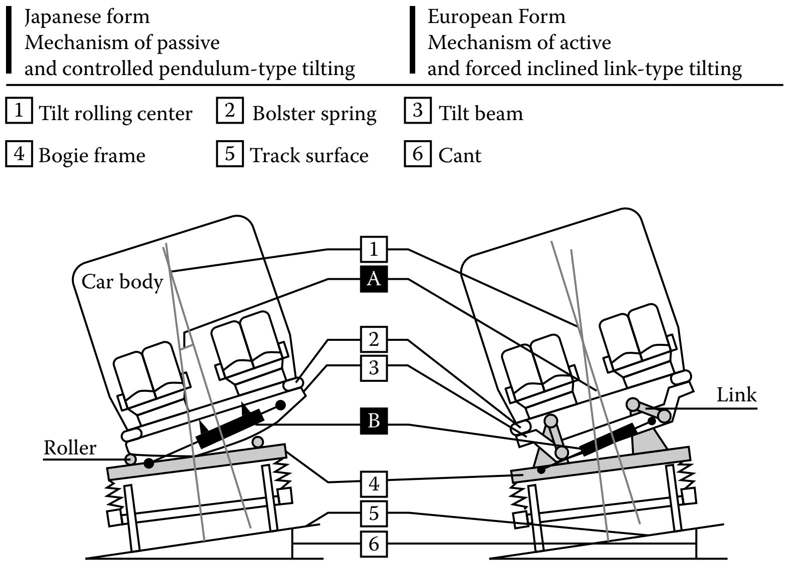

Figure 13.4 The two, active tilting techniques. (Adapted from Online image, available at: http://www.hitachi-rail.com/products/rolling stock/tilting/feature05.html (accessed December 2013). From Hitachi.)

to its speed, with the aid of a system that enables continuous communication between the track and the train. This technique could be considered more efficient than the previous one because of the improved lateral dynamic comfort experienced by the passengers when entering and exiting a curve. This comes as a result from the prediction of the exact point when the tilting commences. The above advantage is offset by the disadvantage of requiring installation of equipment on the track. This system was manufactured by Hitachi, and allows for a tilting angle of $\varphi_t = 5°$.

Figure 13.6 showcases the classification of tilting trains based on the tilting operation principle, the tilting technique and the tilting system that is being used.

Figure 13.5 Taiwan tilting train. (Adapted from Online image, available at: http://www.hitachi.co.jp/New/cnews/month/2015/01/0109.html (accessed October 2015). From Hitachi.)

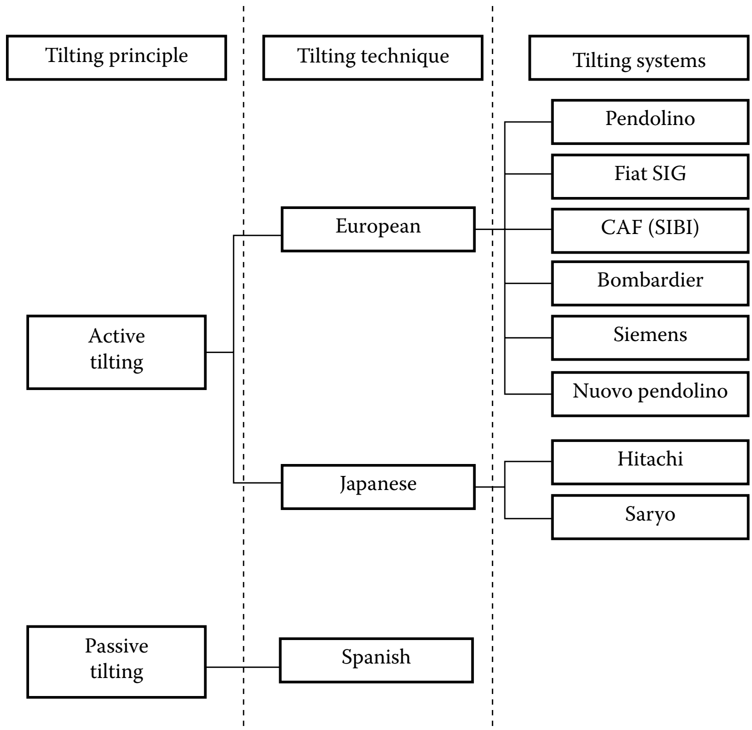

Figure 13.6 Classification of tilting trains based on the tilting operation principle, the tilting technique and the tilting system that is being used.

13.3 MAIN CONSTRUCTIONAL AND OPERATIONAL CHARACTERISTICS OF TILTING TRAINS

13.3.1 Performances in terms of speed

Tilting trains are operated on new high-speed tracks where their maximum running speed can be $V_{max} = 220–320$ km/h; however, most commonly they are operated on upgraded conventional-speed tracks where their maximum running speed is $V_{max} = 150–200$ km/h. On metric gauge tracks, the recorded running speed varies between $V_{max} = 120$ and 165 km/h, and the maximum speed is observed at the Australian railway network ($V_{max} = 165$ km/h).

Equation 13.1 is the mathematical expression of the tilting train's speed when running on curved segments of the horizontal alignment for standard gauge track (Chiara et al., 2008):

$$V_p = \sqrt{[(U + I + 2 \cdot e_o \cdot \tan\varphi_t) \cdot R_c]/11.8} \tag{13.1}$$

where

V_p: passage speed at curves (km/h)
R_c: radius of curvature in the horizontal alignment (m)
U: cant of the track (mm)
I: cant deficiency (mm)
φ_t: tilting angle (degrees)
$2e_o$: theoretical distance between the running surfaces of the right and the left wheel when centred ≈ distance between the vertical axis of symmetry of the two rails

Equations 13.2 through 13.4 provide the mathematical expression of speed obtained by applying Equation 13.1 for conventional trains, trains with passive tilting and trains with active tilting, respectively, considering the following track and rolling stock data:

$2e_o = 1{,}500$ mm
$U = 160$ mm
$I = 105$ mm ($\gamma_{nc} = 0.7$ m/s^2)
$\varphi_t = 0^o$ for conventional trains
$\varphi_t = 3.5^o$ for tilting trains with passive tilting
$\varphi_t = 8^o$ for tilting trains with active tilting

$$V_p = 4.74\sqrt{R_c} \tag{13.2}$$

$$V_p = 5.49\sqrt{R_c} \tag{13.3}$$

$$V_p = 6.35\sqrt{R_c} \tag{13.4}$$

Figure 13.7 illustrates a diagram which provides the variation of permitted passage speed V_p at curves in relation with the radius of curvature R_c for the three aforementioned train categories.

According to Equations 13.2 through 13.4 and Figure 13.7, tilting trains allow for an increase of running speed by around 15% in the case of passive tilting and around 35% in the case of active tilting. For the same geometrical design features, active tilting achieves higher running speeds by about 15% than those of passive tilting.

In practice, tilting trains have recorded increased running speeds in horizontal curves by 10%–40%.

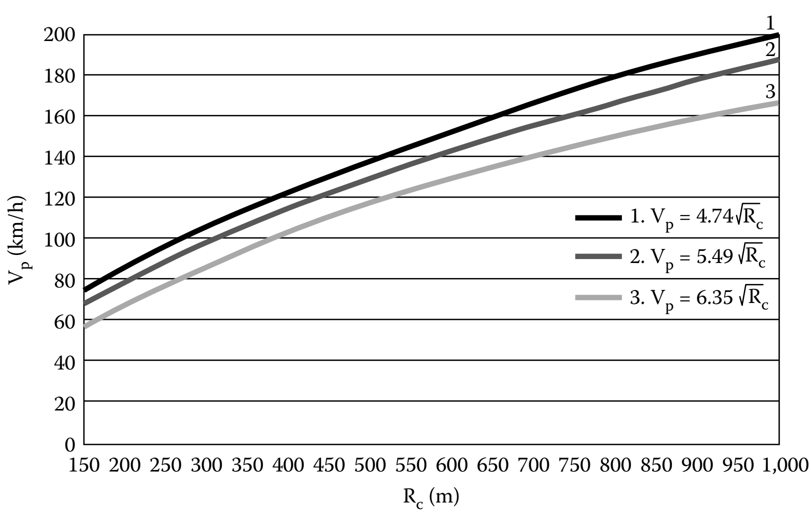

Figure 13.7 Variation of the permitted passage speed at curves in relation to the radius of curvature, for conventional trains and for tilting trains. (Adapted from Chiara, B., Hauser, G. and Elia, A. 2008, I Treni ad Asseto Variabile: Evoluzione, Prestazioni e Prospettive, *Ingegneria Ferroviaria*, July–August (7–8) 609–648.)

13.3.2 Tilting angle

Trains with active tilting achieve tilting angles of up to 8° (8° using the European technique and 5°–6° using the Japanese technique). This results to an elevation of the car body in curves at the level of the external rail up to 200 mm for standard gauge track.

Trains with passive tilting achieve tilting angles between 3° and 5° (3.5° under service conditions).

13.3.3 Track gauge

Tilting car-body trains can run on various track gauges (1,067, 1,435, 1,524 and 1,668 mm). In Spain, some Talgo trains are equipped with wheelsets that have variable axle length.

13.3.4 Axle load

It usually ranges from 14 to 17 t. In conventional high-speed trains, axle load reaches 17 t, while that for conventional-speed trains is higher.

13.3.5 Track superstructure

As mentioned in Section 13.1, the speed increase affects the track's superstructure as it increases the value of lateral forces acting on the contact surface between the wheel and rail (Kubata, 1992; UIC, 2003; Pyrgidis, 2009). Hence the track should be able to offer increased lateral resistance. To meet these requirements, the track must have rails that are continuous welded with a minimum weight of 60 kg/m (UIC 60), concrete sleepers and elastic fastenings. Slab track is the optimal solution. In the case of ballasted track, a ballast of great hardness and minimum layer thickness of 35 cm is needed (with an increased degree of compaction and increased ballast shoulder width). Last but not least, the track should be well stabilised.

13.3.6 Bogies technology

All available bogies' technologies can be used for tilting trains (conventional bogies [Pendolino system], bogies with self-steering axles [ASEA system] and bogies with independently rotating wheels [Talgo system]).

13.3.7 Train formation

Tilting trains serve passenger trips exclusively. As regards their formation, tilting trains can be loco-hauled trains, railcars (multiple units) or push–pull trains.

13.3.8 Signalling

Tilting car-body technology is compatible with all signalling systems.

13.3.9 Traction

Tilting trains can be powered by either diesel or electricity. In the case of electric power, the electric power supply system requires adjustments in accordance with the technical specifications of the tilting train.

13.3.10 Cost of rolling stock supply

The cost of tilting trains is around 15% higher than the cost of conventional trains (Profillidis, 2004).

13.4 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

Every railway company wishes to reduce travel times and increase dynamic passenger's comfort for the network it manages. Operating tilting trains is an alternative to having a higher quality infrastructure that ensures the desired performances described above.

Within this framework, the selection dilemma between conventional trains and tilting trains is directly related to the infrastructure in which they will be circulated.

More specifically, tilting trains can be a choice in case of the following.

13.4.1 Existing conventional-speed infrastructure

When the track alignment is characterised by a large percentage of horizontal curved segments and the travel times need to be reduced without intervention on the track's layout geometric characteristics (radii, transition curves, and cant of track). As a prerequisite, the track's superstructure must be in good condition and it must have the necessary mechanical strength so as to be able to receive increased lateral loads exerted on the interface of the wheel flange and the rail. Meanwhile, the vehicles' new kinematic gauge, as a result of the tilting, must still lie within the limits of the current allowable minimum structure gauge.

Table 13.3 summarises the advantages and disadvantages of operating tilting trains on existing conventional-speed infrastructure.

The reduction in travel time is directly related with the length of horizontal curved segments.

Table 13.4 shows

- For routes where the curved segments of the horizontal alignment amount to 50%, 60%, 70% and 80% of the total route length
- Considering an increase of speed at curved sections of the track equal to 25%, by using tilting technology

the percentage (%) reduction of travel times obtained by using tilting technology trains compared with conventional ones.

Table 13.3 Advantages–disadvantages of tilting trains in comparison with conventional trains–operation on existing conventional-speed infrastructure

Advantages	*Disadvantages*
• Reduction of running time on curved segments of the track • Reduction of the total travel time • Zero cost of interventions on the track's alignment • Maintenance of mixed train traffic operation	• Additional stress and wear on the track's superstructure • Increased cost of rolling stock acquisition • Potential cost for the improvement of the track's lateral resistance and for widening the structure's gauge • More frequent maintenance of the track's superstructure • More complex maintenance of rolling stock

Table 13.4 Percentage reduction of travel time by using tilting trains in comparison with conventional ones, for various percentages of horizontal curved segments' length in the total route length

Percentage of curved segments of the horizontal alignment in the total path length	50%	60%	70%	80%
Percentage reduction of travel time by using tilting trains	10%	12%	14%	16%

In practice, the reduction in travel times is around 12%–20%.

Operating tilting trains allows for maintaining the mixed traffic of trains. More specifically, it allows for routing of passenger and freight trains on the same track, since no modifications on the track's alignment geometry is required (see Chapter 17).

When tilting trains equipped with conventional bogies run on tracks with small horizontal alignment radii (R_c = 300–500 m) and where flange contact is occurred, then, according to the literature (Pyrgidis, 2009):

- A 10% increase in passage speed V_p results in a 0.5 t increase of the guidance force F_{11} (front wheelset, left wheel) (Figure 13.8)
- A 20% increase in passage speed V_p results in a 1.0 t increase of the guidance force F_{11}
- A 30% increase in passage speed V_p results in a 1.5 t increase of the guidance force F_{11}

When tilting trains equipped with conventional bogies run on tracks with medium horizontal alignment radii (R_c = 1,200–2,000 m) where flange contact is avoided, then an increase in energy consumption at the level of four wheels of each bogie is observed.

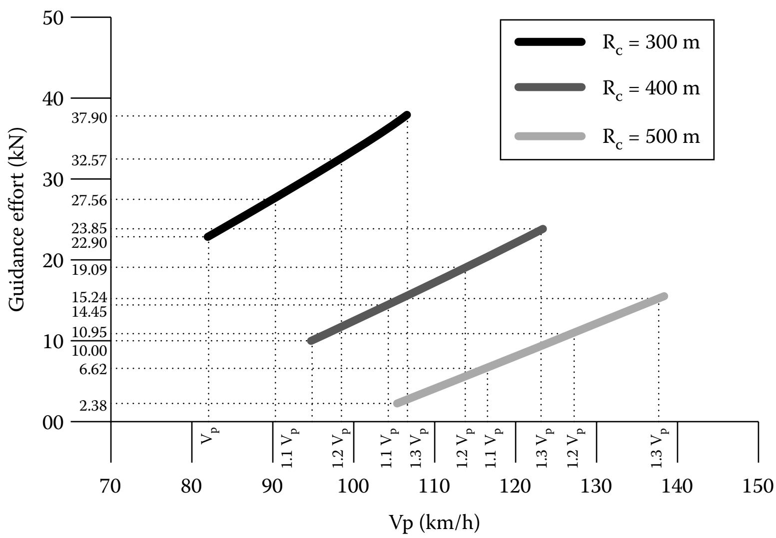

Figure 13.8 Motion of tilting trains with conventional bogies on horizontal curves with small radius – variation of the guidance force in relation to passage speed. (Adapted from Pyrgidis, C. 2009, Tilting trains/conventional trains – Comparison of the lateral forces acting on the track, *Ingegneria Ferroviaria*, April 2009, Rome, No 4, pp. 361–371.)

13.4.2 New conventional-speed infrastructure

When the terrain requires the adoption of many horizontally curved segments with small radius and the aim is to achieve better performance in terms of travel times or dynamic passenger comfort.

The use of bogies with self-steering axles for tilting conventional-speed trains is the preferred choice for relatively low running speeds (V_{max} < 160 km/h) and for large numbers of curved segments of the horizontal alignment with small and medium radii (Pyrgidis and Demiridis, 2006; Pyrgidis, 2009).

13.4.3 New high-speed infrastructure

When the route is characterised by a high percentage of curved segments of the horizontal alignment and the aim is to further increase the lateral dynamical passenger comfort.

For high-speed tilting trains, it is essential to use conventional bogies or bogies with independently rotating wheels (Pyrgidis and Demiridis, 2006; Pyrgidis, 2009).

During the tests, but also during the up-to-date operation of tilting trains the following malfunctions were recorded and resolved:

- 'Locking' of the car body in a tilted position on straight path, that is, after leaving the curved segments
- Problems during coupling of railcars (multiple units)
- Disabling of the tilting mechanism due to bad weather conditions
- Compatibility problems with the signalling system
- Problems with software
- Very intense tilting of the car body

13.5 HISTORIC OVERVIEW AND PRESENT SITUATION

Table 13.5 summarises the available data for tilting trains operating worldwide. All the data recorded relate to the year 2014. The raw data were obtained from various available sources and cross checked. Afterwards they were further manipulated for the needs of this chapter.

Tilting car-body train technology has been a research topic of interest for railway engineers for decades.

In more detail:

1930s

Experiments commence in the United States. The vehicles are named 'pendulum' and achieve the tilting of the car body through inertia forces.

1950s

In 1956, in France, the engineer Mauzin builds and experiments with a single vehicle which uses passive (unassisted) tilting.

1960s

In 1967, the Italian company Fiat Ferroviaria manufactures the prototype tilting train YO160, named Pendolino.

In 1968, the United States launches the first high-speed tilting train with passive tilting (TurboTrain).

Active tilting trains are being studied in Germany for the first time.

Table 13.5 Tilting trains operating worldwide (indicative data)

Type	*Operation launch year*	*Country*	*Tilting system*	*Max speed (km/h)*	*Track gauge (mm)*	*Train formation*	*Tilting angle (degrees)*
TurboTrain	1968–82	Canada	UAC (Passive)	193	1,435	LC + TC	
381 Series	1973	Japan	Hitachi	120	1,067	EMU	5
Talgo Pendular	1974–93	Spain–Germany	TALGO	180–220	1,668–1,435	LC + TC	3–3.5
ETR 401	1975	Italy	Pendolino	250	1,435	LC + TC	13
Class 370 APT	1975	United Kingdom	Pendolino	200	1,435	LC + TC	
LRC	1982	Canada	Bombardier	160	1,435	LC + TC	4–5
TRD 594	1982	Spain	CAF (SIBI)	160	1,668	DMU	6
ETR 450	1988	Italy	Pendolino	250	1,435	EMU	8
2000 Series	1989	Japan	Hitachi	130	1,067	DMU	5
ICT	1989	Germany	Pendolino	230	1,435	EMU	8
X 2000	1990	Sweden	ASEA	210	1,435	LC + TC + DVT	8
Series 8000	1992	Japan	Hitachi	160	1,067	EMU	5
VT 610	1992	Germany	Pendolino	160	1,435	DMU	8
8000 Series	1992	Japan	Hitachi	160	1,067	EMU	5
E351 Series	1993	Japan	Hitachi	130	1,067	EMU	5
ETR 460	1994	Italy	Pendolino	250	1,435	EMU	8
HOT 7000	1994	Japan	Hitachi	130	1,067	DMU	5
83 Series	1994	Japan	Hitachi	130	1,067	EMU	5
281 Series	1994	Japan	Hitachi	130	1,067	DMU	5
961 Series	1995	Japan	Hitachi	130	1,067	EMU	5
283 Series	1996	Japan	Hitachi	145	1,067	EMU + DMU	5–6
383 Series	1996	Japan	Hitachi	130	1,067	EMU	5
SM 220-SM3	1996–2003	Finland	Pendolino	220	1,524	EMU	8
ETR 470	1996	Switzerland	Pendolino	200	1,435	EMU	8
VT 611	1997	Germany	Adtranz	160	1,435	DMU	8
ETR 480	1997	Italy	Pendolino	250	1,435	EMU	8
BM 73	1997	Netherlands	ASEA	210	1,435	EMU	8
QR Tilt Train	1998	Australia	Hitachi	165	1,067	EMU + DMU	5
ALARIS 490	1999	Spain	Pendolino	220	1,668	EMU	8
VT 612	1999	Germany	Adtranz	160	1,435	DMU	8
CP 400	1999	Portugal	Pendolino	220	1,668	EMU	8
ICN	2000	Switzerland	Adtranz	200	1,435	EMU	8
ACELA	2000	United States	Bombardier	240	1,435	DMU	4.2 (4–6)
SZ 310	2000	Slovenia	Pendolino	200	1,435	EMU	8
TALGO 350	2000	Spain	TALGO	350 (330)	1,435	LC + TC	3.5
BM 93 (Talent)	2001	Netherlands	Bombardier	140	1,435	DMU	7
Class 221 Super Voyager	2002	United Kingdom	Bombardier	200	1,435	DMU	

(*Continued*)

Table 13.5 (Continued) Tilting trains operating worldwide (indicative data)

Type	*Operation launch year*	*Country*	*Tilting system*	*Max speed (km/h)*	*Track gauge (mm)*	*Train formation*	*Tilting angle (degrees)*
British Rail CLASS 390	2003	United Kingdom	FIAT SIG	225	1,435	EMU	8
CDT 680	2005	Czech Republic	Pendolino	230	1,435	EMU	8
MEITECHU Series	2005	Japan	Nippon Sharyo	120	1,067	EMU	5
N 700	2007	Japan	Hitachi + Nippon Sharyo	300	1,435	LC + TC	5
ETR 600	2008	Italy	Pendolino (New)	250	1,435	EMU	8
ETR 610	2008	Switzerland	Pendolino (New)	250	1,435	EMU	8
E5 Series Shinkasen	2011	Japan	Hitachi	300	1,435	EMU	5
E6 Series Shinkasen	2013	Japan	Hitachi	320	1,435	EMU	5
TALGO AVRIL	2013	Spain	TALGO	380	1,435	LC + TC	5
TRA	2013	Taiwan	Nippon Sharyo	150	1,435	EMU	5
TTX (Hanvit 200)	2013	South Korea	KRRI	200 (180)	1,435	EMU	8
ED250	2014	Poland	Pendolino	250	1,435	EMU	8

DMU, diesel multiple units; EMU, electrical multiple units; LC, locomotive; TC, trailer vehicle; DVT, driving van trailer.

1970s

The first experimental tilting trains using active tilting feature in Japanese railways.

Tilting trains conquer Italy with Pendolino train at first and ETR trains later on.

Development of tilting trains' technology in Spain (Talgo).

The British railways build the experimental tilting train named APT (advanced passenger train). The technology used for the construction of the train is similar to that of the Italian ETR and the Spanish Talgo. The British railways, due to technical problems, were never able to make this train reliable enough to be put into service.

ABB develops an alternative active tilting system.

1980s

In Italy, commercialisation of tilting trains is intensified (ETR trains).

Operation of tilting trains in Japan and Canada (LRC).

1990s

In the early 1990s, the Swedish train X2000 (manufactured by ABB) is put into service.

In 1992, tilting trains enter the German market with the VT 610, followed by Finland and Switzerland (1996).

In 1996, the French National Railways (SNCF) and Alstom study the use of tilting trains on existing tracks, and the conversion of a TGV into tilting TGV.

In 1997, the application of tilting technology on the connection Boston–Washington is studied.

The decade 1990–2000 is the peak period for this technology. During this decade most of the tilting body train models were built. After 2000, the number decreases, yet it remains constant.

2000s

In 2000, Alstom purchases 51% of Fiat Ferroviaria's shares. In 2002, Alstom purchases the total of Fiat Ferroviaria's shares.

In 2001, ADtranz is purchased by Bombardier Transportation. During the time of purchase, ADtranz was the second manufacturer of rail rolling stock worldwide.

In 2004, Virgin Trains puts tilting trains named British Rail Class 390 into service on the main line of the UK's west coast.

2010s

In December 2013, Talgo manufactures AVRIL (Alta Velocidad Rueda Independiente Ligero) train, with design speed V_{rs} = 380 km/h.

In South Korea, the Korea Railroad Research Institute (KRRI) together with a team of Korean manufacturers built the Tilting Train Express (TTX), or else Hanvit 200.

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Chapter 14

Metric track gauge interurban railway networks

14.1 DEFINITION AND DESCRIPTION OF THE SYSTEM

The term ‘metric track gauge railway networks’ describes networks with a track gauge of 900–1,100 mm.

Metric gauge appears in many countries, mostly in parts of their network; but in some cases it is the exclusive track gauge.

The principal difference between normal and metric track gauge lies in the permanent way. Metric track gauge occupies less surface area, which means lower implementation cost and less expropriation. Moreover, a metric gauge line is generally quite flexible and adaptable to the landscape; this feature makes it ideal for mountainous areas.

The vast majority of metric tracks were designed and constructed many years ago. The following were mainly the reasons for choosing a narrower track gauge back then:

- *Economic*: It is estimated that a metric gauge track results in 30% less implementation cost than the track of normal gauge.
- *Strategic*: To ensure incompatibility with the networks of neighbouring countries, and thus protection from a potential enemy invasion.
- *Operational*: In those days, traffic volumes and transport capacity requirements were quite low, and the performance of the rolling stock was limited.

In this framework, most metric lines are nowadays characterised by

- Low running speeds, usually $V_{max} < 120$ km/h
- Limited axle loads, usually $Q_{max} < 16–18$ t
- Small transport capacity, passenger or freight

Nevertheless, there are lines that operate at a maximum running speed of $V_{max} = 160$ km/h (South Africa), axle loads of $Q = 25–30$ t (Australia, South Africa) and transport more than 6 billion passengers annually (Japan, JR East).

The tilting train of Queensland Rail in Australia holds the record of being the fastest train on metric track gauge in the world ($V = 210$ km/h).

Metric gauge systems are classified as follows:

On the basis of the exact track gauge:

- 914 mm (3 ft)
- 950 mm
- 1,000 mm (metric)
- 1,050 mm and 1,055 mm
- 1,065 mm and 1,067 mm (Cape gauge)

On the basis of the maximum running speed V_{max}:

- Low speed ($V_{max} \leq 80$ km/h)
- Medium speed (80 km/h < $V_{max} \leq 120$ km/h)
- High speed ($V_{max} > 120$ km/h)

On the basis of the maximum allowed axle load Q_{max}:

- Small axle load ($Q_{max} \leq 16$ t)
- Medium axle load (16 t < $Q_{max} \leq 20$ t)
- High axle load (heavy haul) ($Q_{max} > 20$ t)

14.2 GENERAL OVERVIEW OF METRIC GAUGE INTERURBAN RAILWAY NETWORKS

Metric track gauge applies for nearly all railway systems (urban, suburban, interurban, steep longitudinal gradients, etc.). This chapter examines only interurban systems according to their definition which was given in Chapter 11.

The overall line length of metric railway networks worldwide exceeds 200,000 km, accounting for approximately 20% of the total line length.

The above data and the data recorded and analysed in the following relate to the year 2014. The raw data were obtained per country, from various available sources and cross-checked. Afterwards they were further manipulated for the needs of this chapter.

Table 14.1 presents the countries that operate metric lines, with their respective lengths and track gauges. The content of the aforementioned table is constrained to countries with interurban metric lines with a network length of at least 100 km.

Table 14.1 Length of metric lines per exact track gauge and per country

Gauge 1,067 mm		**Gauge 1,055 mm**		**Gauge 1,000 mm**	
Angola	2,638 km	Algeria	1,085 km	Argentina	11,080 km
Australia	15,160 km	**Total length**	**1,085 km**	Bangladesh	1,830 km
Costa Rica	350 km			Bolivia	3,600 km
Ecuador	965 km	**Gauge 1,050 mm**		Brazil	23,489 km
Honduras	785 km	Jordan	507 km	Cambodia	612 km
Indonesia	5,961 km	Syria	327 km	Chile	2,923 km
Japan	20,264 km	**Total length**	**834 km**	China	466 km
Malawi	797 km			French-speaking African countries	10,700 km
Mozambique	2,983 km	**Gauge 950 mm**		Germany	140 km
Namibia	2,382 km	Erythrea	118 km	Greece	961 km
New Zealand	3,900 km	Italy	345 km	India	6,000 km
Nicaragua	373 km	Somalia	114 km	Malaysia	1,677 km
Taiwan	1,097 km	**Total length**	**577 km**	Myanmar	3,200 km
Tanzania	2,600 km			Portugal	141 km
Zambia	1,870 km	**Gauge 914 mm**		Spain-FEVE	1,250 km
Zimbabwe	3,400 km	Canada	523 km	Spain-FGC	140 km
Total length	**65,525 km**	Colombia	3,154 km	Switzerland	624 km
		El Salvador	555 km	Thailand	4,431 km
Gauge 1,065 mm		Guatemala	332 km	Tunisia	1,762 km
South Africa	33,520 km	**Total length**	**4,564 km**	Vietnam	2,600 km
Total length	**33,520 km**			**Total length**	**77,626 km**

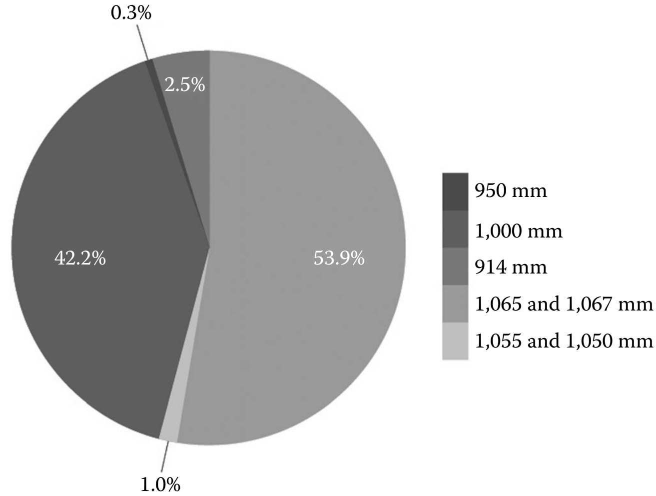

Figure 14.1 Distribution of metric track lines by exact track gauge.

Figure 14.1 illustrates the percentage distribution of metric lines by exact track gauge, according to the classification presented in Section 14.1.

Figure 14.2 presents the percentage distribution of metric lines by continent.

From Table 14.1 and Figures 14.1 and 14.2, it is evident that

- South Africa possesses the longest metric railway network (33,520 km), followed by Brazil (23,489 km), Japan (20,264 km), Australia (15,160 km) and Argentina (11,080 km).
- The majority of metric lines (54.9%) feature a gauge of 1,065 mm and 1,067 mm.
- The largest percentage of metric lines is in Africa (34.8%).
- Many neighbouring countries use the same track gauge, which ensures interoperability.

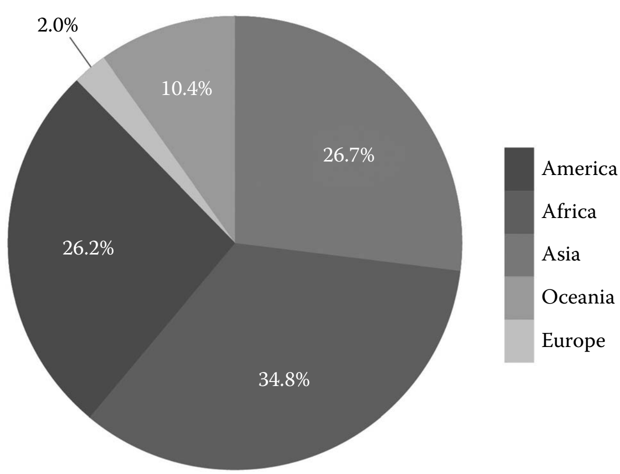

Figure 14.2 Distribution of metric track lines by continent.

The current trend dictates that metric lines are converted to normal gauge lines. On the contrary, normal gauge lines are not converted into metric anywhere in the world, while as for the ones already existing are only conditionally upgraded.

14.3 MAIN CONSTRUCTIONAL CHARACTERISTICS OF INTERURBAN METRIC TRACK GAUGE LINES

14.3.1 Track alignment: Differences between tracks of metric and normal gauge

The track geometry alignment design of a metric track (1,000 mm) is different from a normal track (1,435 mm) in the following aspects (Montagné, 1988; OSE, 2004):

- The distance between the vertical axis of symmetry of the two rails ($2e_o$) in the metric track is smaller (1,056 mm, for 31.6 kg/m rails) than in a normal track (1,500 mm), since it depends directly on the track gauge.
- For the same speed V and curve radius R_c of the horizontal alignment of a metric and a normal gauge track, the value of the theoretical cant U_{th} is about 30% smaller for the metric track than it is for the normal one.

For $g = 9.81$ m/s^2, $2e_o = 1{,}056$ mm and after appropriately converting the relevant units, the following mathematical relation is derived:

$$U_{th}(\text{mm}) = 8.306 \cdot \frac{V_{max}^2(\text{km/h})}{R_c(\text{m})} \tag{14.1}$$

Equation 14.1 holds for rails that weigh 31.67 kg/m. In case of rails that are of different quality, a different coefficient (than 8.306) is used. For instance, for UIC 50 and UIC 54 rail type, the coefficient is set to 8.416.

- For the same speed V and curve radius R_c of the horizontal alignment of a metric and a normal track, the value of the cant U is smaller in the case of the metric track.
- The maximum value of cant deficiency I for metric track gauge lines does not exceed 100 mm, while for the normal track gauge lines one can find values of up to 150 mm.
- The metric track, when considering the same daily traffic volume, speed and curve radius in the horizontal alignment, is designed with a smaller value of cant excess E_c than the normal gauge track. Specifically, for the metric track the maximum allowed cant excess usually does not exceed 70 mm, while a normal gauge network can allow values of up to 110 mm (UIC Meter Gauge Group, 1998).
- The rate of change of cant deficiency for a metric track is theoretically smaller than for a normal gauge track, regardless of the applied speed.
- Metric track lines are usually designed with shorter transition curves than normal gauge lines; in some cases transition curves are completely omitted, leading to higher twist values. The same problem appears when a metric track applies the same value for change of cant per unit length as a normal track. Therefore, the maximum allowed twist in the metric track is lower than in the normal track (Rhätische Bahn, 1986; SNCF, 1998). In the horizontal alignment, when designing successive compound or reverse curves, the intermediate straight segment can be shorter in the case of a metric track.

- In the vertical alignment, when designing successive gradient curves, an intermediate straight segment of minimum 20 m must be included in the metric track (30 m for the normal gauge track).

In general, there are no commonly accepted regulations or standards regarding track layout and track superstructure characteristics of a metric line.

Table 14.2 presents in summary the main track alignment geometric characteristics, the boundary values and standards used for the Greek metric railway network. These values originate from the 'Regulation for the Track Layout and Superstructure of a Metric Line', which was composed in 2006 in order to upgrade the network and increase the track design speed from $V_{max} = 90$ km/h to $V_{max} = 120$–140 km/h (OSE, 2004; Lambropoulos et al., 2005).

The minimum allowed horizontal curve radius for newly constructed main lines is set to $R_{cmin} = 150$ m. The minimum length for transition curves can be calculated from the following two equations (both must be fulfilled):

$$L_{k\,min} = 0.4 \cdot U \tag{14.2}$$

$$L_{k\,min} = \frac{U \cdot V_{max}}{125} \tag{14.3}$$

where

$L_{k\,min}$: minimum allowed length for a transition curve (m)
U: cant of the track (mm)
V_{max}: maximum running speed (km/h)

Table 14.2 Greek railway network of metric track gauge – track alignment geometric characteristics

Characteristics	*Notation*	*Value*
Track gauge	$2e$	1,000 mm
Distance between the vertical axis of symmetry of the two rails	$2e_o$	1,056 mm
Maximum cant	U_{max}	105 mm
Maximum cant deficiency	I_{max}	75 mm
Maximum cant excess	E_{cmax}	70 mm
Maximum rate of change of cant deficiency	$\Delta I_{max}/\Delta t$	40 mm/s
Maximum track twist	g_i	$\frac{125}{V_{max}}$ (mm/s)
Highest permitted value for track twist	g_{imax}	2.5 mm/m
Maximum permitted residual lateral acceleration	γ_{ncmax}	0.07 g
Minimum horizontal alignment curve Radius	R_{cmin}	$R_{cmin}(m) = 0.04615 \cdot V_{max}^2(km/h)$ $R_{cmin}(m) = 0.05728 \cdot (V_{max}^2 - V_{min}^2)\,(km/h)$
Minimum vertical alignment curve radius	R_{vmin}	2,000 m – Convex curves 1,500 m – Concave curves 500 m – Auxiliary or siding lines
Maximum longitudinal gradient	i_{max}	1.6%
Cant (normal)	U	$U\,(mm) = 4.845 \cdot \frac{V_{max}^2\,(km/h)}{R_c(m)}$
Theoretical cant	U_{th}	$U_{th}\,(mm) = 8.306 \cdot \frac{V_{max}^2\,(km/h)}{R_c(m)}$

Table 14.3 Metric gauge railway networks in Australia, South Africa, Japan, Switzerland and France – track alignment geometric characteristics

Network operator	*QR, Spoornet, JR East*	*RhB*	*SNCF*
Track gauge (2e)	QR: 1,067 mm Spoornet: 1,065 mm JR East: 1,067 mm	1,000 mm	1,000 mm
Minimum horizontal alignment curve radius (R_{cmin})	100 m	45 m	
Straight segment length between reverse curves	≥10 m	≥20 m	≥30 m
Theoretical (U_{th}) and normal cant (U)		$U = \frac{4.55 \cdot V_{max}^2}{R_c}$	$U = \frac{8.5 \cdot V_{max}^2}{R_c}$
Maximum cant deficiency (I_{max})	50–60 mm	86 mm	70–90 mm
Maximum normal cant (U_{max})	100–110 mm	105 mm	100 mm
Highest permitted value for track twist (g_{imax})	QR: 3.33‰ Spoornet: 4.0‰ JR East: 2.5‰	2.5‰	2.5‰
Maximum longitudinal gradient (i_{max})	QR : 20‰ Spoornet: 25‰	70‰	25‰
Minimum vertical alignment curve radius (R_{vmin})	$R_{V_{min}} = 2{,}000$ m–4,000 m ($V_{max} > 100$ km/h) $R_{V_{min}} = 1{,}650$ m ($V_{max} > 60$ km/h) Convex: $R_{V_{min}} = 525$ m Concave: $R_{V_{min}} = 300$ m ($V_{max} < 60$ km/h)	Convex: $R_{V_{min}} \geq \frac{V_{max}^2}{2.5}$ $R_{V_{min}} = 1{,}500$ m Concave: $R_{V_{min}} \geq \frac{V_{max}^2}{4}$ $R_{V_{min}} = 1{,}000$ m	$0.35 \cdot V_{max}^2$ $R_{Vmin} = 2{,}000$ m

In parts of the line that are being renovated or upgraded, the constant 115 is used instead of 125, where possible. Moreover, the coefficients 0.04615 and 0.05728 of Table 14.2 adopted for the calculation of R_{cmin} are valid for the case of continuous welded rails that weigh 31.6 kg/m ($2e_o = 1{,}056$ mm). For rails of type UIC 50 and UIC 54, the aforementioned coefficient values change to 0.04675 and 0.05804, respectively.

Table 14.3 provides the equivalent standards for the networks of Australia (QR), South Africa (Spoornet), Japan (JR East), Switzerland (RhB) and France (SNCF).

14.3.2 Track superstructure

Table 14.4 presents the characteristics of the track superstructure (track design speed, axle load, and track panel components) for several metric gauge networks around the world.

In most cases, the maximum permitted axle load lies in the range of Q = 14–18 t. However, some metric networks, such as in South Africa (Spoornet) and in Australia (Queensland Rail), can sustain axle loads that exceed 25 t, owing to the reinforced track structure.

The majority of small axle-load metric networks operate on rails of specific weight of 30–50 kg/m and length of 10–20 m.

For high speeds (V > 120 km/h) and axle loads Q > 14 t it is generally necessary to use rails that weigh more than 31.6 kg/m while as for the use of the lighter rail that weighs 31.6 kg/m the good condition of the track's geometry is a prerequisite (IVT/ETH, 1994).

Table 14.4 Track superstructure characteristics from different metric gauge networks around the world

		Speed		*Load*	*Rails*			*Sleepers*							
								Type				*Dimensions*			*Distance in between (mm)*
Network operator	*Country*	*Passenger trains (km/h)*	*Freight trains (km/h)*	*Per axle (t)*	*Weight (kg/m)*	*Length (m)*	*CWR*	*Wooden*	*Monoblock concrete*	*Twin block*	*Steel*	*Length (mm)*	*Width (mm)*	*Height (mm)*	
Spoornet	South Africa	160	80	30	57	36	Yes	Yes	Yes			2,200	300	258	650
QR	Australia	140	100	25	60		Yes	Yes	Yes			2,150	230	115	685
JR Hokkaido	Japan	140	100		50	25	Yes		Yes			2,000	240	240	640
JR West	Japan	130	110	18	50	25	Yes		Yes			2,000	240	170	568
JR East	Japan	130	110	16	50	25	Yes	Yes	Yes			2,000	200	140	625
JR Shikoku	Japan	130	95	16	50	25	Yes		Yes			2,000	240	201	661
JR Kyushu	Japan	130	95		50	25	Yes	Yes	Yes			2,100	200	140	550
SNCFT	Tunisia	120	65	16	46	18	Yes			Yes	Yes	1,711	274	200	600
SRT	Thailand	120	70	15	40	18	Yes	Yes				2,000	200	150	650
CJR	Japan	120			60	25	Yes		Yes			2,000	240	190	
SARCC	South Africa	110	80	22	48	36									700
JR Freight	Japan	–	110	17	50	25	Yes	Yes				2,100	200	140	694
Perumka	Indonesia	100	75	18	42	17	Yes	Yes	Yes			2,000	220	200	600
KTMB	Malaysia	100	72	16	40	12	Yes	Yes	Yes			2,000	280	235	600
IR	India	100	75		41	13	Yes				Yes				650
OSE	Greece	90	75	14	31	15	Yes	Yes				1,800	220	130	640
TAZARA	Tanzania	80	80	20	45	12	Yes	Yes	Yes			2,000	200	145	680
SITARAIL	Côte d' Ivoire	80	60	17	36		Yes			Yes	Yes	1,800	120	75	666
PNR	Philippines	75	45	15	37	20	Yes	Yes	Yes			2,134	203	127	600
KR	Kenya	72	65	17	40	12	Yes				Yes	2,057	279		500
URC	Uganda	65	55	14	40	12	Yes	Yes			Yes	1,990	280	90	700
RNCFM	Madagascar	60	45	16	30	12	Yes	Yes			Yes	1,900	220	170	600
CFL	Angola	50	45	16	30	10					Yes	1,850			700
CFRC	Cambodia	45	30	15	30	12	Yes	Yes			Yes	1,800	200	150	650

Source: Adapted from Paradissopoulos, I. and Paradissopoulou, F. 1998, Comparative analysis of track technical characteristics of metric gauge railway networks, *Technika Chronika*, TEE, I, 1.

Regarding the sleepers, the use of wooden sleepers is widely applied in low-speed metric gauge networks, while the use of concrete monoblock sleepers applies in high-speed metric gauge networks. According to the UIC recommendations, reinforced concrete sleepers are to be used in cases of track design speeds of $V_d = 120–160$ km/h and axle load $Q \leq 13$ t. Moreover, the sleepers should be placed at a distance of ≤650 mm from each other. The length of the sleepers in most metric gauge tracks of small axle load ranges from 1,800 to 2,000 mm, and the distance between sleepers varies between 600 and 700 mm.

The use of sleeper anchors (the so-called 'safety caps') drastically increases the lateral resistance of the track panel; however, this also depends on how dense they are installed (i.e. every fourth, third, second or even every sleeper). According to Lymberis et al. (2006), it is feasible to apply continuous welding of the track by using concrete sleepers; also feasible is the placement of anchors on each sleeper and track that weigh 31.6 kg/m at a radius of $R_c = 100$ m.

Most metric tracks employ ballasts of 240–300 mm thickness, but there are some cases where the ballast is constrained to 180–210 mm (SNCF, 1998).

14.4 ADVANTAGES AND DISADVANTAGES OF INTERURBAN METRIC GAUGE LINES

Advantages

- The integration of a metric gauge line to the terrain requires smaller width for the permanent way when compared with the normal gauge line. This results in lower implementation and expropriation costs.
- It is relatively easy and also environmentally friendly to lay the track in areas with difficult landscape.

Disadvantages

- There is a higher risk of vehicles' overturning, which inevitably limits the allowed running speed (Chenuc, 1988). Reducing the speed results in the reduction of track capacity (Pyrgidis and Stergidou, 2011).
- The dynamic gauge of vehicles running on metric tracks is normally smaller than on normal tracks to ensure stability. This limits the capacity of freight wagons and imposes constraints on carrying large cargo and on the comfortable arrangement of the interior of passenger rolling stock.

14.5 REQUIREMENTS FOR IMPLEMENTING THE SYSTEM

As already mentioned, there is a trend toward converting metric gauge lines into normal ones (while no normal gauge line is being converted into narrow gauge one).

The construction of a new metric line should be carried out when

- Interoperability with the neighbouring networks is guaranteed. This condition explains why particular track gauges are widespread in discrete parts of the planet.
- It is considered to be an extension of an existing metric gauge line which is desired to be retained and upgraded.
- The regions served are environmentally sensitive areas and/or with a difficult landscape.
- It constitutes a section of an isolated network, thus making it economically more feasible.

Figure 14.3 Mixed gauge track (normal and metric gauge tracks, Kiato, Greece). (Photo: A. Klonos.)

In all cases, transition from a metric track to normal gauge or the opposite is a costly and timely process. This can be achieved in several ways

- By transferring passengers/cargo at the transition point. However, this has a negative impact on the cost and transport time.
- By use of a mixed gauge track (Figure 14.3). This option provides the possibility of operating both metric and normal gauge bogies on the same track superstructure. Nevertheless, the implementation and maintenance costs in this case are significantly high (Alvarez, 2010).
- By changing bogies. In this method, the car body is lifted while the bogie underneath changes. The car body is then repositioned to the new bogie. Evidently this approach is considerably time-consuming.
- By use of varying-gauge axles (automatic track gauge changeover). In this case railway vehicles need to be equipped with wheelsets of varying gauge (Figure 14.4). The gauge of these axles' changes, following the transition as the train passes over a section of the track panel that is specifically equipped with a relevant track gauge change over configuration (Figure 14.5). In this procedure, the wheels of the vehicles 'unlock' and move closer to each other – or further, depending on the case – and 'relock' accordingly,

Figure 14.4 Varying-gauge wheelset. (Online image, available at: https://www.youtube.com/watch?v=U_LFIUkcPNM, Talgo 250 gage change – Animated (accessed 14 March 2015).)

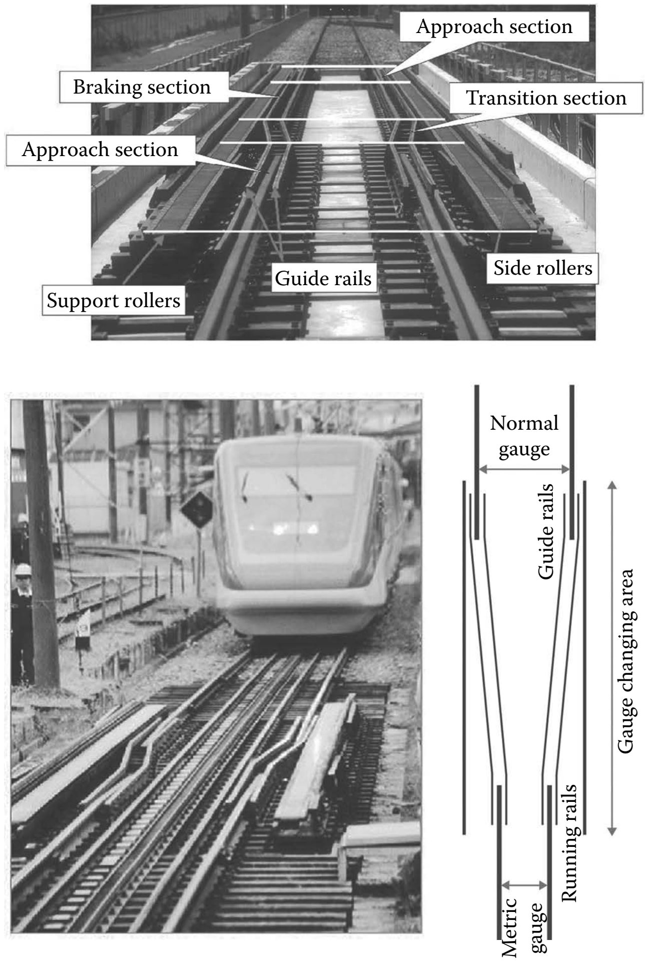

Figure 14.5 Track gauge change over equipment. (Adapted from Motoki, H. et al., 2001, *Developing a gauge-changing EMU*, available at: http://www.uic.org/cdrom/2001/wcrr2001/pdf/sp/1_1_1/413.pdf.)

while the train runs over the specified section. The transition mechanism of the axle ensures the safe standstill of the wheel. Such systems are being used in several countries (Spain, France, and Japan) and mostly around cross-border railway passes of neighbouring countries with different track gauges (e.g., border pass Spain–France). Such a novel system based on independently rotating wheels is now under development in Switzerland to be used in Montreux–Oberland–Bernois network allowing through services between Interlaken and Montreux with change of gauge taking place in Zweisimmen (Prose AG, 2015).

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Chapter 15

Organisation and management of freight railway transport

15.1 SERVICES AND CARGO MOVEMENT

In freight railway transportation, there are differences in the provided services which can be attributed to the variety of customer demands and, most importantly, to the volume and type of the transported cargo (Alias, 1985).

The provided services fall into the following categories:

- Less-than-wagonload services
- Wagonload services
- Services of combined transportation
- Services of heavy haul transport
- Special services

The less-than-wagonload services, also termed as part-load traffic, LCL (less-than-carload), LTL (less-than-truckload) concern the transport of small packages under very strict time conditions and can be characterised as high-speed freight services (e.g., TGV postal high-speed trains) (Troche, 2005; Zanuy, 2013). In the provided rail freight services are also included the transportation of small containers, pallets and other forms of cargos that do not make up a full wagon, under given conditions (Zanuy, 2013).

Wagonload services are distinguished in single wagonload (SWL) services and in train-load services (TL).

SWL services involve cargo movements using single wagon trains. In these traditional freight services, the wagons (each wagon containing loads of single or multiple shipper(s)/consignee(s)) start their journey from different origin points or end their journey at different destinations which necessitates that they pass through intermediary marshalling yards.

Regarding TrainLoad services cargo movement is performed by

- *Unit trains also termed full trains*: These trains are formed of full wagons which start from the same origin and travel to the same destination. These trains make full use of the tractive power of the locomotives. They do not stop at marshalling yards and they run between two specific terminal stations. They have one shipper, one consignee, and one bill of lading and serve more than one final receivers (Zanuy, 2013).
- *Block trains**: Like unit trains, block trains run between two specific terminal stations without stopping at any marshalling yard. Their difference lies in the fact that they do not use all the power of their traction units while hauling.
- *Exclusive trains**: They are unit trains or block trains that serve a single final receiver.

* The nomenclature varies. Block trains and exclusive trains are also usually termed 'unit trains'.

The wagons' assembling process in marshalling yards is time-consuming and costly; hence during the scheduling of the transported freight distribution, particular concern is given so that most wagons are not subjected to the process of sorting more than once, or twice at most.

Combined train transportation covers a wide range of services (see Section 15.4). Heavy haul transportation services use wagons of high transport capacity forming trains with axle load of Q = 25–50 t (see Chapter 16).

Special services involve dedicated connections between mines and ports or mines and factories. The railway operates on a continuous basis ensuring a constant load flow, while special wagons are often used for transportation. A 'door-to-door' delivery can also be considered as a special service, since the railway company undertakes receiving and delivering products from/to the user's 'door'.

15.2 SERVICE LEVEL OF FREIGHT RAILWAY TRANSPORT: QUALITY PARAMETERS

The level of service provided by freight railway transportation is evaluated according to the following quality parameters:

- Tariffs
- Service frequency
- Punctuality in the delivery of goods and regularity of itineraries
- Transport possibilities provided by the railway system (e.g., capability of transporting goods of large quantities, and bulk products)
- Facilities offered at freight stations (e.g., storage area, and appropriate equipment for loading and unloading)
- Flexibility and quick response to customer demands
- Insurance during the transportation of goods (against theft, damage)
- Monitoring of cargo during its movement
- Special services provision (transport of dangerous goods, combined transport, and specialised cargo transport)

Additional quality parameters for international freight railway transport are given below:

- Short delays at border stations
- Ensuring interoperability

Some of the main features that characterise freight railway transport are:

- *Length of trains*: In Europe, the length of trains can reach up to 750 m. In the United States, trains used for the transportation of conventional loads are larger (up to 2 km), whereas in dedicated mine railways, their length can reach up to 4 km for heavy haul rail transport.
- *Weight of trains*: Trains routed in Europe usually weigh around 1,500–2,000 t. Only three countries, namely Russia, Sweden and Norway, operate trains of 5,000 t. The maximum train weight of heavy haul rail transport worldwide reaches up to 34,000 t.

  Both weight and length are important productivity factors for the railway freight industry and are influenced by various features of the system.

The permissible length is dictated by station length and by the design of the brake system. On the other hand, the features of the brake system and the design of the coupling equipment affect the permissible weight.

The adoption of the automatic coupler (Janney/Knuckle coupler in North America and China, Willison/SA3 in former the USSR) allowed for heavier and longer trains while the continued use of the screw coupler in Europe lead to shorter/lighter trains and thus less economical service provision.

- *Maximum train running speed*: It ranges between $V_{max} = 100$ and 120 km/h. The heavy haul rail transport requires lower running speeds ($V_{max} = 80$–100 km/h).
- *Axle load*: For the transportation of conventional loads, the maximum permissible axle load on normal gauge tracks is $Q = 22.5$ t, whereas for the transportation of heavy loads, it is $Q = 25$–40 t.
- *Longitudinal gradient*: Operation is highly influenced by the longitudinal gradient of the track. Gradients that exceed 1.6% result in a restriction of the train's maximum weight and speed while they also increase in the requirements for the rolling stock (available power, braking performance, etc.).
- *Reliability and regularity of services*: The acceptable delay time of a freight train depends on the length of the route. One could consider the acceptable delay time to be equal to 1 h per 500 km of length. Punctuality, in other words, minimisation of delays, constitutes one of the parameters that determines the level of quality that a railway system provides.
- *Terminal layout*: Freight transport requires stations to be equipped with special facilities for collecting and handling goods. The average distance between small marshalling yards in Germany is around 160–200 km, between medium marshalling yards 800–1,200 km and between large marshalling yards 3,000–3,500 km (Jorgensen and Sorenson, 1997).

15.3 SCHEDULING OF FREIGHT TRAIN SERVICES

Contrary to passenger railway services, freight railway services do not usually provide their customers with regular timetables (Lambropoulos, 2004).

The common practice is to provide certain routes in the form of reserved 'paths' which are performed only when there is sufficient load.

Another common practice is that there are no predetermined routes and the trains depart as soon as the load reaches a certain target (e.g., departure as soon as 1,500 t or 100 wagons are gathered). This is practiced by the American railway network. Adjusting and scheduling of services takes place ad hoc on an everyday basis.

However, there is a trend to apply regular services similar to those of passenger trains. The paths are used with or without sufficient cargo and the customers' cargo is shipped with the first available train, the main objective being to increase the transportation speed and the reliability of the railway system.

15.4 COMBINED TRANSPORT

The container revolutionised freight transport since it allows for easy transfers among all modes of transportation. At the same time, other techniques have been developed, allowing the transportation of cargo from one transport mode to another without handling the cargo itself. The common feature of all these techniques is the use of more than one transport modes in an attempt to make use of the unique advantages of each one of them. This is what characterises this type of transportation as *combined transport*.

The potential combinations of various techniques and transport modes are usually grouped into three categories:

- *Multimodal transport*: Transportation is achieved with more than one modes. More precisely, one mode uses the other on a common route (i.e. one mode is loaded on the other, Figure 15.1).

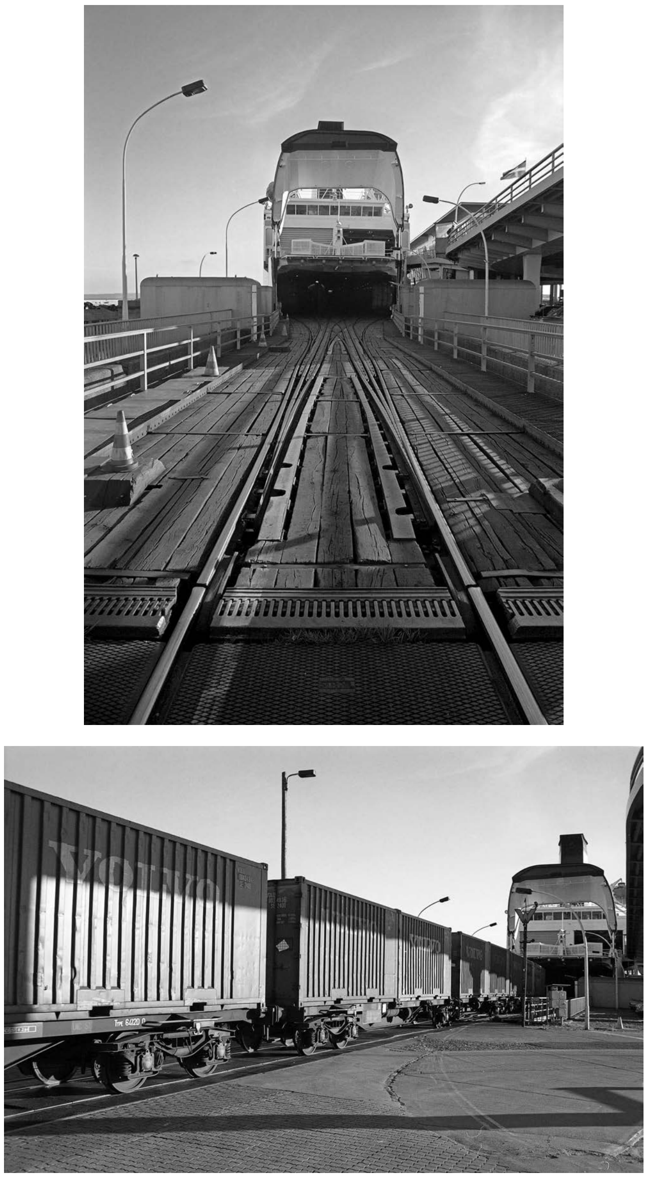

Figure 15.1 Multimodal transport – train loading in ship, Sassnitz hafen, Germany. (Photo: A. Klonos.)

- *Intermodal transport*: The major part of transportation takes place on a ship, whereas distribution/collection of the cargo in ports continues with other transport modes.
- *Combined transport*: The major part of transportation takes place on either railway or barge and collection/distribution continues on the road.

Usually the term *combined* covers all categories, whereas the American term *intermodal* covers train/truck transportation. However, in the European Union (EU), the term *combined transport* refers to the transit of a trailer or semi-trailer truck (with or without tractor), a swap body or a container, using road transportation for the initial or final part of the route and ships, barges or railways for the main part of the route.

The EU has made great attempts to achieve an unimpeded and economically viable transportation of passengers and cargo throughout Europe at the least possible external costs (accidents, pollution, and traffic congestion). The combined transport technique perfectly responds to this policy. The 'door-to-door' cargo transfer can combine different means of transportation: flexible road transport for collection/distribution and – friendly to the environment – sea, inland water and rail transport for the main transportation.

Aside from the above-mentioned trend, the EU is also facing the issue of alpine crossings. Switzerland and Austria pose restrictions to the passage of lorries from their land, either with general restrictions (restriction of trucks weighing more than 28 t) or with technical or economic barriers (tolls imposed according to the technology of the vehicle [emissions, sound pollution, and suspensions friendly to the road surface] restrictions in permits).

Generally these policies aim to promote the use of railways for long-distance transport.

Combined transport constitutes a growing market for American railways since road transport is the country's third best customer after coal mines and power stations. In Europe, though railways receive a great financial aid, their financial return is not very promising. In reality, although a significant part of the European rail freight transport is being handled with combined transport, revenue remains low.

Railways can offer four different 'products' of combined transport:

- *Transport of unified load* is the transport of containers and swap bodies with flat railway vehicles – platforms. Containers are mostly used for sea transport while swap bodies tend to be used for train–truck transport (Figures 15.2 and 15.3).

Figure 15.2 Swap-body loading to a railway wagon, Malmo, Sweden. (Photo: A. Klonos.)

Figure 15.3 Container loading to a railway wagon, Thriasio, Greece. (Photo: A. Klonos.)

- *Piggyback transport* is the transport of a road trailer placed on special wagons – platforms without its tractor unit and its driver (Figure 15.4).
- *Rolling road or rolling highway transport* (Rollende Landstraβe–ROLA): This is the transport of a whole truck on special flat, low-floor railway vehicles. The driver accompanies its truck on a special sleeping car, which is included in the train formation (Figure 15.5).
- *Roadrailer technique* is the technique in which railway bogies are adjusted on the basis of special road trailers (with their rubber tyred wheels) which can be hauled like railway wagons. It is the least common technique in Eurasia.

Table 15.1 presents the advantages and disadvantages as well as the technical and operational requirements of the above techniques.

Figure 15.4 Piggyback rail transport, Gavle, Sweden. (Photo: A. Klonos.)

Figure 15.5 Technique of rolling highway transport (ROLA), Bockstein, Austria. (Photo: A. Klonos.)

Table 15.1 Combined transport techniques: Advantages/disadvantages

Techniques	*Advantages*	*Disadvantages*	*Requirements*
Unified loads (containers)	• Standardised technique • Global dissemination • Possibility of stowage in height • Robust construction, durability	• Large stowage area needed • Expensive handling equipment	• Special rail vehicles • Cranes for container handling • Applied mainly for sea transport
Unified loads (swap bodies)	• Large capacity • Technology affordable for end users • Easier handling	• Lighter structure • Large stowage area needed	• Cranes for container handling with special equipment • Applied mainly for land transport
Piggyback (transport of truck trailer placed on special wagons)	• Fast upload • Flexibility	• Large area needed • Enlarged loading gauge • High cost of equipment • Special management/ organisation	• Cranes for container handling • Special wagons (trailer of flat cars [TOFC]) • Applied mainly for land transport
Rolling road transport (ROLA)	• Simple procedures to be followed at stations • No additional investment by road transport companies is required	• Expensive and maintenance heavy rail vehicles • Low speeds • High operational cost • Enlarged loading gauge/ increased loads	• Special low-floor railway vehicles • The driver has to accompany the train • Applied mainly for land transport

15.5 MASS TRANSPORT

The term 'mass transportation' is used to describe (Pyrgidis, 1989; Savy, 2006)

- The transport of large quantities
- The long-distance transportation

- The high-speed transportation
- The transportation with modes of great transportation capacity
- The transportation with transport means of high occupancy

The policy of mass transport somehow justifies the role of transhippers since they can achieve 'economies of scale' for a specific transfer. On the other hand, the additional profit obtained by transshipments allows consignors and consignees to pay a smaller fare for their cargo to be transported. Finally, the 'small' customers who fail to gather large loads can also join in mass transport, thus reducing the cost of transferring their cargo.

According to the above, consignees, consignors and shippers have common interests in this policy of mass transportation. As a result, research have been conducted and money has been invested on means of transportation, techniques and equipment suitable for massification of flows. This has resulted in the development of

- Container ships
- Cargo planes
- Road trucks of heavy tonnage
- Special trains for moving cargo (unit trains, block trains)
- Automated marshalling yards

The railway is a suitable means for mass transportation. According to the volume of load it carries, the railway adopts different solutions in order to respond more successfully to the policy of mass transport (Figure 15.6) (Pyrgidis, 1989).

More precisely:

a. When freight units are too big, cargo movement is performed with unit trains or block trains which are not passing through marshalling yards. For this purpose, special sidings are used to connect the railway line with the production centres or the production centres are installed/set up very close to the line.
b. When freight units are relatively big, expediting is done with single wagon trains that move at high running speeds (V_{max} = 120 km/h) while their delay in marshalling yards is the least possible (automatic marshalling yards).
c. Finally, in the case of medium- and small-size freight units (5–10 t) expediting is done with single wagon trains but the service level is low.

Nowadays consignors and consignees have three main demands:

- Immediate order delivery, which requires frequent and flexible services
- 'Stock' reduction; this implies that only the ordered products will be produced
- A wide variety of every product

To address these demands, the following four solutions are used today in the field of freight railway transport:

- Warehouses (platforms)
- Multifunctional railway stations
- Freight transport villages
- Combined transport

Warehouses ensure mass transportation for small loads and also reduce the transportation cost for both shippers and consignors. After the cargo has been gathered in a specific

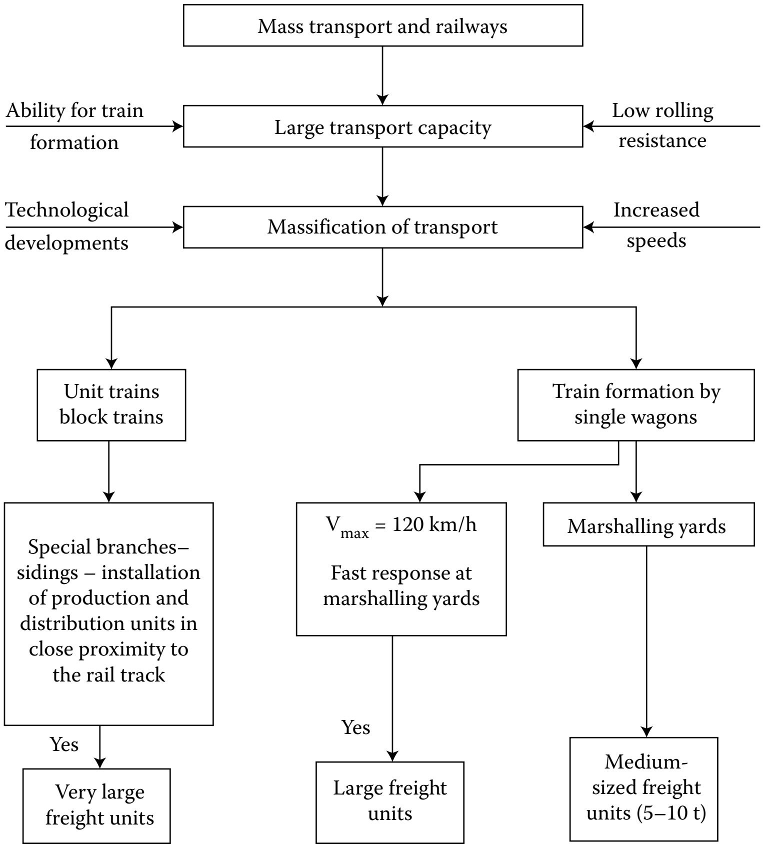

Figure 15.6 Mass transportation and railways. Feasible operational alternatives. (Adapted from Pyrgidis, C. 1989, *Transport de mercaderies: Avantages i inconvenients de la massificacio de fluxos*, Espais – revista del department de politica territorial i ombres publiques de Catalunya, No. 17, May–June 1989, pp. 26–31.)

place, it is then channelled to various nearby destinations by means of small trucks thus responding to the deadlines of product delivery. This process within the chain of transportation improves the service level and reduces the 'logistic cost'.

Freight transport villages are designated areas where all activities regarding transport logistics and distribution of goods are performed by various handlers. Their operation supports the transport of goods especially in big urban centres. Freight transport villages connect different ways of transport, give access to transport corridors and offer telematics services. Their main goal is to relieve big urban centres of the traffic congestion caused by trucks. The companies related to the freight transport villages have offices in these centres equipped with modern advanced technologies, providing customers with a number of facilities.

Railways now tend to organise a hub-and-spoke system in order to rationalise both the unit/block train product and the single wagonload traffic.

The concept is to establish a network of

- Marshalling yards (the hubs of the system)
- Satellite freight stations and freight villages near these stations

- Smaller freight stations between the marshalling yards which can provide adequate cargo to justify the costs of wagon shunting

Between the hubs run mainly unit or block trains. Only limited number of trains perform pick or leave wagons at intermediate stations.

The trains are marshalled at the yards and arranged into groups for the satellite stations, freight villages or sidings.

The aim is to utilise the maximum available capacity of a train path (in terms of tractive power, length and weight) between the marshalling yards and to reduce the delay times at intermediate stations.

15.6 TRANSPORTATION OF DANGEROUS GOODS

The term 'dangerous goods' covers materials and objects, the transportation of which is allowed only under certain conditions. These loads are categorised according to their physical and chemical properties. Fluid and solid fuel, gas, explosives, nuclear material, polluting and corrosive materials are considered as dangerous loads.

Transportation of the aforementioned loads is done with various specialised wagons (e.g., tank wagons, closed vehicles and open and side dump vehicles with high walls).

Cargo shipments are done with unit and block trains and combined transport techniques.

Dangerous cargo transport differs from other freight activities in the following (OSE, 1998):

- Procedures of product identification and completion of consignment
- Wagon loading and unloading
- Regulations for dangerous load trains parked in high-risk areas
- Signage of wagons
- Design and method of storage/packaging
- Emergency response
- Procedures for train shunting, train formation and braking
- Cleaning of empty wagons

Activities related to transporting these products are legally established by international conventions and are conducted under strictly defined conditions of safety. The regulations applied to rail transport are COTIF/CIM/RID (www.cit-rail.org/en/rail-transport.../cotif/).

Policies of dangerous loads railway transportation must abide by the following (Journal de la marine Marchande, 1994; Duche and Baspeyras, 1996; Pyrgidis and Basbas, 2000; Basbas and Pyrgidis, 2001).

15.6.1 Differentiation from the rest of freight transport services

Organisation and execution of different activities associated with the transportation of dangerous loads should be examined and treated separately from the transport of conventional (non-dangerous) loads.

15.6.2 Creation of safe transport conditions

In case of an accident the extent of damage can be relatively higher than that caused by any other modes of transportation, because of the massification of railway transport. Therefore, prevention of any incident is of the utmost importance.

In this context, it is required that

- The condition of track superstructure used is really good (to avoid derailment).
- There is a special maintenance programme and testing of the rolling stock (to avoid derailment, material leakage, etc.).
- The maintenance and repairing area of vehicles transporting dangerous goods is different from that of the conventional wagons; special instructions for the repairing of vehicles are made and special areas are provided for cleaning empty tanks (to avoid explosions, fainting and fumes).
- Reception and distribution tracks specifically assigned to trains transporting dangerous goods are equipped with proper fencing, lighting, fire extinguishing and sewage systems (protection of high-risk areas).
- Modern technologies are applied for the monitoring of loads during the whole route (for immediate intervention and effective treatment).

15.6.3 Special measures to protect the environment

An accidental leakage of the vehicle tanks or the transported goods' packages, or even the maltreatment of the material throughout the loading and unloading-transfer phase, involves a high risk of polluting the environment due to the nature of these products (see Section 19.4). Measures taken for accident prevention also ensure the safety and protection of the environment. However, there are additional measures, aiming exclusively at the protection of the environment. These are

- The drainage system of waste at cleaning stations of empty tanks
- The special design of the superstructure (e.g., slab track instead of ballasted track) at points of shunting tracks used by trains carrying dangerous loads, aiming at the protection of the groundwater in case of leakage

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Chapter 16

Heavy haul rail transport

16.1 DEFINITION AND GENERAL DESCRIPTION OF THE SYSTEM

The term 'heavy haul rail transport' describes any railway operation using trains of a minimum axle load equal to 25 t (Q = 25–40 t).

Trains that are intended for carrying heavy loads operate either in dedicated freight railway corridors or in corridors with mixed traffic operation. They serve the transport of conventional and hazardous goods. Railway networks intended for heavy haul transport satisfy all categories of cargo movement and usually have a broad or normal track gauge (Figure 16.1). The heavy haul rail transport requires reduced running speeds (V_{max} = 80–100 km/h).

Table 16.1 presents, for various heavy haul railway corridors, the number of tracks and the track gauge used, the length of the connection they serve, the allowed axle load, the total weight of the train, the traffic volume that they transport annually, the features that are selected for the track superstructure and the maximum running speed of freight trains.

In Section 16.3, constructional and functional characteristics of heavy haul rail transport systems that differ from those of conventional freight rail systems are identified. The heavy haul transport differs from other rail freight activities and should be considered and treated separately from the transportation of conventional loads.

16.2 THE INTERNATIONAL MARKET IN HEAVY HAUL RAIL TRANSPORT

Heavy trains are commonly used in America, South Africa and Australia, where heavy haul rail transports have the largest share in the rail freight market. The train that carries the heaviest loads worldwide is also the longest one and is found in Australia. This train's payload is 82,000 t, its gross weight is 99,734 t and it is composed of 682 wagons and 8 diesel locomotives of 6,000 HP. The train's length is 7.2 km and it transports bulk minerals from one part of Australia to the other across thousands of kilometres, passing through uninhabited areas and deserts (Christogiannis, 2012).

In the EU, the average hauled weight of freight trains reaches 1,000 t, while in America trains that are 10–20 times heavier are used, the axle weight of which is double than the respective axle weight of trains at EU countries; trains operated in America also are 2–3 times longer than EU ones. In the EU, trains with large axle load have been used for experimental route tests, but in practice such trains have only been launched in Sweden. Their weight approaches 8,160 t (gross weight), and the line where they are used has a length of 43 km; however, this length represents only 0.3% of the total length of the EU railway networks (Christogiannis, 2012).

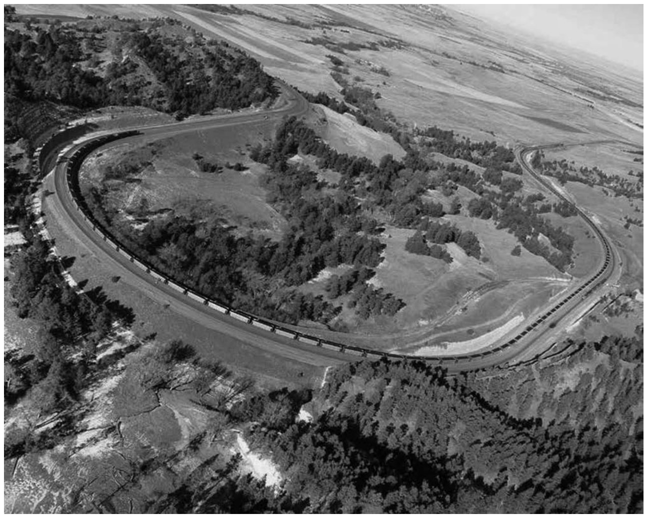

Figure 16.1 Heavy haul rail transport, Convoy formation. (Adapted from Vossloh 2014, Vossloh_Segment_ Heavy Haul_09-2014_ENG.PDF.)

The tracks that are used for the heavy haul rail transport either belong to mixed traffic operation networks (India, Russia, etc.), in which the tracks are designed for a high design axle load, or are networks where freight trains have priority which are specially constructed for heavy loads (United States, Canada, etc.).

Countries in which such transportation is widely used are usually countries where the largest volume of transported goods are bulk products, which are related either to the needs of power production, for example, carbon, or to their manufacture, for example, iron ore. In the EU on the contrary, the transported products are mainly construction materials and chemicals, and for such products there is high competition between the railway and maritime and road transport.

16.3 DIFFERENCES BETWEEN CONVENTIONAL AND HEAVY HAUL FREIGHT RAILWAY NETWORKS

The operation of heavy haul freight trains on the one hand seems to achieve 'economies of scale' as these trains have a much higher transport capacity than the conventional freight trains. On the other hand, however, such an activity increases the implementation and maintenance cost of the railway track. The reason for this is that many features of the heavy haul freight trains/wagons differ substantially from the respective features of the conventional haul freight trains/wagons.

In columns (1)–(3) of Table 16.2, the features that differentiate between the two systems are presented.

16.4 IMPACTS OF HEAVY HAUL RAIL OPERATIONS AND MAIN DESIGN PRINCIPLES

Column (4) of Table 16.2 lists the impacts, both positive and negative, which cause differentiations between the two systems, as well as the requirements imposed on their design, construction and operation (Pyrgidis and Christogiannis, 2013).

Table 16.1 Main characteristics of railway lines intended for heavy haul rail transport (indicatively)

Country/line	*Line length (km)*	*Number of tracks/ gauge (mm)*	*Axle load (t)*	*Train load (t)/annual payload (million t)*	*Type of carried cargo*	*Rail's weight/ sleepers type*	*Maximum running speed (km/h)*
Australia/Pilbara–Rio Tinto	1,100	Single/1,435	32.5	29,500/220	Steel	68 kg/m/concrete	80
South Africa/Sishen–Saldanha	861	Single/1,067	30	34,000/47	Steel	60 kg/m/concrete	80
North America/Burlington	43	Double/1,435	30	19,000/50	Coal	68 kg/m/wooden and concrete	75
Norway–Sweden/Ofotbanen	43	Single/1,435	31	8,200/30	Steel	54–60 kg/m/ wooden and concrete	50–90
Russia–Mongolia–China–Kazakhstan/Trans-Siberian	9,244	Single/1,520	>30	>30,000/60	Containers	68–75 kg/m/ wooden and concrete	80
United States/Wyoming Joint Line–Powder River Basin	165	Double/1,435	>30	19,000/300	Coal	68 kg/m/concrete	80
China/Datong–Qinhuangdao	653	Double/1,435	25	20,000/380	Coal	75 kg/m/concrete	80–100
Western Australia/Fortescue	280	Single/1,435	40	32,000/55	Steel	68 kg/m/concrete	80
NW Australia/Hamersley	388	Single/1,435	30	26,000/55	Coal, minerals, iron, and steel	68 kg/m/concrete	75

Table 16.2 Features of the heavy haul freight wagons/trains that differ significantly from those of the conventional haul freight wagon/trains – effects (positive or negative) and requirements for the design, construction, operation and maintenance of the railway system

Wagon/train characteristics that are different in the two railway systems (1)	*Conventional freight trains values of the features (2)*	*Heavy haul freight trains values of the features (3)*	*Effects (positive or negative)/requirements for heavy haul railway networks (4)*
Running speed (V_{max})	60–120 km/h (100 km/h)	50–100 km/h (80 km/h)	*Effects*: Lower track capacity, longer travel time *Requirements*: Smaller curvature radius in the longitudinal and vertical alignment
Axle load (Q)	16–25 t	25–40 t	*Effects*: Higher track geometry defects deterioration rate, longer train braking distance, and higher transported volume of goods *Requirements*: Steeper gradients in vertical alignment, heavier rails, sleepers of higher mechanical resistance, thicker ballast layer, longer signal spacing, greater traction power requirements, higher maintenance needs, and wagons of higher transport capacity
Train weight (B_{tr})	1,500–3,000 t	5,000–35,000 t	*Effects*: Greater braking weight, higher transported volume of goods *Requirements*: Steeper gradients in vertical alignment, longer signal spacing, and greater traction power requirements
Daily traffic load (T_f)	10,000–100,000 t	100,000–300,000 t	*Effects*: Higher track geometry defects deterioration rate, and higher transported volume of goods *Requirements*: Heavier rails, sleepers of higher mechanical resistance, thicker ballast layer, and higher maintenance needs
Train length (L_{tr})	400–800 m	1,000–4,000 m	*Effects*: Lower track capacity, higher transported volume of goods *Requirements*: Longer tracks and platforms in stations
Vehicle clearance gauge	Normal	Enlarged	*Effects*: Larger rolling stock gauge *Requirements*: Differentiates depot and station dimensioning, distance between track centres, and height clearance under civil engineering structures
Type of transported goods	All types	Mainly steel, ore and coal	

Source: Adapted from Pyrgidis, C. and Christogiannis, E. 2013, Freight dedicated railway corridors for conventional and for heavy loads – A comparative assessment of the economic profitability of the two systems, 3rd International Conference on Recent Advances in Railway Engineering – ICRARE 2013, Iran University of Science and Technology, 30 April – 1 May, 2013, Teheran, Congress Proceedings.

The axle load is directly or indirectly involved in the analytical expressions of all forces acting on the wheel–rail contact surface and influences the behaviour of both the rolling stock and the track (Pyrgidis, 2009).

The presence of high axle loads creates many technical problems. The transporting of heavy loads results in increased stresses applied on the rails and transferred by the features of the track superstructure to the subgrade. This has serious implications on the rails (such as cracks, damage to the weld points and breakage due to fatigue) and on the sleepers (e.g., wear). Moreover, damage is observed at the areas of switches and crossings. Stresses on the track bed layers and the formation layer are likely to exceed the permissible values. Therefore, the maintenance cost and the frequency of track inspections are increased.

In order for the heavy haul rail transport to become financially efficient, it is required: (i) for heavy loads' lines that are already in operation to be frequently checked concerning defects of the track superstructure and (ii) for lines that are under construction, all features of the infrastructure to be specially designed. More specifically, as discussed below, the main requirements are the use of heavier rails, the use of sleepers that are of greater mechanical strength, the increased density of the sleeper layout, the special design of the ballast regarding its thickness and the type of materials used, the provision of specifically designed formation layer and the improvement of the soil if it is of poor quality.

For heavy haul rail transport rail ($Q > 30$ t) the specifications and technical solutions that are described hereunder should be adopted.

16.4.1 Selection of track infrastructure components

16.4.1.1 Selection of the track's alignment geometric characteristics

In case of broad gauge track, the same track infrastructure can serve much heavier loads than in the cases of normal and metric gauge track.

In practice, an increase in the axle load (up to a certain degree) is not treated by widening the gauge but by increasing the mechanical strength of the features of the track's superstructure (e.g., heavier rails).

The increase of the axle load leads to the adoption of lower longitudinal gradients. The adoption of larger longitudinal gradients in many cases reduces the tunnelling works and the construction of bridges considerably; however, it requires locomotives of greater traction power (Geo-Technical Engineering Directorate, 2007).

16.4.1.2 Selection of rails

When the axle load is increased, an increase of the cross section of the rail is required, that is rails that are heavier and of greater mechanical strength are required (UIC 60, UIC 72 grade steel 90 kp/mm^2).

Based on the mathematical equation (16.1) (Esveld, 1989), it is deduced that axle loads of 20, 25 and 30 t require rails of minimum weight equal to 50, 60 and 70 kg/m, respectively. An increase of the axle load by 25% requires a 20% increase of the rail weight:

$$B_r = 2.25Q + 3 \tag{16.1}$$

where

B_r: rail weight per metre (kg)

Q: axle load (t)

For axle load up to 25 t rails of 60 kg/m 90 UTS (ultimate tensile strength) is sufficient. For higher axle loads, heavier rails whose cross sections are heavier are required.

For a load Q = 30 t, rails of 68.5 or 71 kg/m must be used.

Regarding the stress that is developed on the rail (and therefore regarding the rail's wear) it increases as the quantity $(Q_o)^v$ increases (Alias, 1977), where ν is exponent, the values of which are between 3 and 4, and Q_o is the wheel load.

Longer wavelength irregularities, usually known as 'waves', which appear on the rail's rolling surface are due to fatigue of the wheel–rail contact surface. During the heavy haul rail transport, the pressure on the contact surface is very high, and this results in the development of corrugations troughs (with gross plastic flow), which have a wavelength of 200–300 mm and a frequency of 30 Hz, for average traffic speed (Grassie and Kalousek, 1993; Grassie and Elkins, 1998). The longer wavelength corrugations have a particularly big impact on the maintenance, as they increase costs by up to 30%.

In any case, it is evident that the increase of axle loads intensifies the phenomena of fatigue and their consequences.

Increased axle load results in wear and fatigue during the early stages of the line's operation (Lichtberger, 2005).

Regarding the frequency of the required track maintenance work, the influence of vertical loads is catalytic. The deterioration of the track's quality is proportional to the third power of the value of the axle loads. An increase in the axle load by 10% reduces the intervals between two consecutive maintenance works by 30% (Liu, 2005).

Finally, with regard to the quality of steel, it is required that the steel used be harder and heat treated, in order to be able to bear the increased loads, to have increased resistance against wear and fatigue and to ensure increased lifetime for the rails.

16.4.1.3 Selection of the type of sleepers and the distances between them

The tracks on which high axle loads are expected to be imposed are usually constructed using sleepers made of prestressed concrete. The sleepers' density is 1,540 sleepers per km or 1,660 sleepers per km. The use of 60 kg/m rails and prestressed concrete sleepers placed at distances of 43 cm between them is suitable for the operation of axle loads that are equal to 30 t.

16.4.1.4 Selection and dimensioning of track bed layer features

By applying the mathematical equation (16.2) (Esveld et al., 1989; UIC, 2006; Profillidis, 2014) the impact of the axle load on the thickness of the track ballast and the sub-ballast layers can be assessed as

$$e_{bt} = e_b + e_{sb} = E_b + a_b + b_b + c_b + d_b + f_b \tag{16.2}$$

where

e_b: ballast layer thickness (m)
e_{sb}: sub-ballast layer thickness (m)
e_{bt}: total thickness of ballast and sub-ballast layers (m)
E_b: parameter that depends on the quality category of the soil and the bearing capacity of the substructure
a_b: parameter that depends on the classification of the track in the UIC classes
b_b: parameter that depends on the sleepers' length and material
c_b: parameter that depends on the volume of the required track maintenance work
d_b: parameter that depends on the maximum axle load Q
f_b: parameter that depends on the track design speed V_d and the bearing capacity of the substructure

By using the mathematical equation (16.2), the contribution rate of the parameter d_b on the total thickness e_{bt}, and, thus, the influence rate of the axle load Q on it can be derived graphically (Figure 16.2). As shown in Figure 16.2 the influence rate of the axle load on the total thickness of ballast and sub-ballast layers is negligible for passenger trains ($Q \leq 20$ t), while for freight trains it can increase the thickness by 10%–21%.

The thickness of the sub-ballast can range between 15 and 75 cm, depending on the quality of the soil material of the substructure.

For an axle load equal to $Q = 30$ t, the required thickness of the ballast is 25 cm and the required thickness of the sub-ballast is 15 cm for speed up to 100 km/h. However, a clean ballast layer with a thickness of 300 mm may prove to be the best solution.

The gravel should be of high hardness. Frequent monitoring of the ballast with the use of mechanical means and appropriate techniques is required.

The existence of high axle loads in combination with a weak subgrade requires that a sub-ballast layer which is no less than 1 m thick be placed between the ballast and the subgrade.

16.4.1.5 Construction principles of the formation layer

The introduction of an axle load of 30 t requires the provision of a formation layer of sufficient thickness in order to improve the bearing capacity just below the sub-ballast. It is obvious that a weak subgrade will lead to a rapid deterioration of the track's geometry, which will render the operation of heavy axle-load trains unsafe, thereby imposing an additional requirement for increased and more frequent maintenance (Dingqing and Chrismer, 1999).

For heavy haul rail transport the following are required:

- Stabilisation of the substructure's soil with suitable mechanical means during construction
- Improvement of the foundation soil in case it is considered to be of poor quality

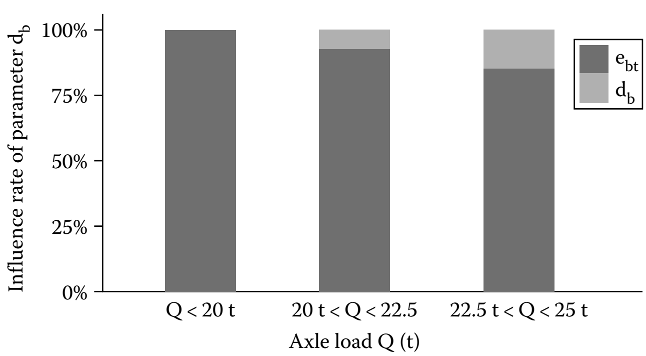

Figure 16.2 Influence of the parameter d_b on the total thickness of ballast and sub-ballast layers e_{bt}. (Adapted from Christogiannis, E. 2012, Investigation of the impact of traffic composition on the economic profitability of a railway corridor – Fundamental principles and mathematical simulation for the selection of operational scenario for a railway corridor, PhD thesis (in Greek), Aristotle University of Thessaloniki, Thessaloniki, Greece; Pyrgidis, C. and Christogiannis, E. 2012, The problem of the presence of passenger and freight trains on the same track, *International Congress TRA (Transport Research Arena) 2012, 'Sustainable mobility through innovation'*, 23–26 April 2012, Athens; Pyrgidis, C. and Christogiannis, E. 2011, The problem of the presence of passenger and freight trains on the same track and their impact on the profitability of the railway companies, *9th World Congress on Railway Research 'Meeting the challenges for future mobility'*, 22–26 May 2011, Lille, France.)

16.4.1.6 Dimensioning of bridges

Bridges require special design for the case of heavy haul rail transport. Since sometimes cracks are developed on the concrete along the bridges it is necessary that the substructure of the bridge be strengthened (see Section 2.2.3.4).

16.4.1.7 Dimensioning of the signalling system

The heavy haul rail transport results in an increase of the total weight of the train, thus increasing significantly the braking distance (which depends on the total weight of the train, the speed of the train, the total resistance of the train and the longitudinal gradient of the line). For this reason there is an urgent need for a special study of the signalling system.

16.4.2 Effects on the rolling stock

The increase of the axle load results in

- An increase of the static gauge of the rolling stock
- An increase of the train's movement resistance

The effect of the rolling stock's static gauge on the components of the railway system mainly concerns the required distance between track centres, the height clearance from the civil engineering structures and the geometrical adequacy of tunnels.

The increase of the train's movement resistance results in an increase of the required engine power of the locomotives. Therefore, special rolling stock is required for the heavy haul rail transport.

More specifically

- The use of vehicles that are made by a material of increased strength is required. The vehicles' design must be characterised by a constant effort to increase transport capacity. At the same time, it must be lightweight in order to be able to move at steep gradients and to develop higher speeds on straight paths.
- As an alternative strategy, the use of wagons with high payload in comparison with their tare (high payload – tare rate) and the increase of the number of axles can also be considered. A variation of the dimensions of railway vehicles and more specifically the reduction of the wheels' diameter is essential.
- The use of 3-axle bogies increases the transport capacity, keeping the forces applied on the track within the permissible limits.

Special probes for the impact of high load on the wheels (wheel impact load detector – WILD) can provide control and monitoring capabilities regarding the effects of heavy rail vehicles on the track, thereby constituting a valuable tool for the study of a heavy haul rail system.

16.4.3 Effects on the operation

The heavy haul rail transport regards trains of large length, thereby reducing the track capacity.

In addition, the heavy haul rail transport usually concerns the transporting of goods over long distances, that is, it is implemented at lines that are of long length. A typical example is Trans-Siberian line, with a length of 9,244 km, which crosses two continents, namely Europe and Asia.

The distributed power, or otherwards the use of more than one locomotive in the train formation (at the front and at the back, at the front and at the middle, and at the front, at the middle and at the back), make the transport operation much more effective. This option reduces stress on couplers and buffers, enables longer and heavier trains, improves force distribution in curves and enables quicker brake response (Hoffrichter, 2014).

The increase of the axle loads from 20 to 30 t is expected to result in an increase in the track's maintenance cost by 3 times, depending on the quality of the infrastructure. It is still unclear as studies show that after the rails are subject to grinding processes and lubrication, the observed increase in maintenance cost is only 3%.

According to the literature (Christogiannis, 2012), in case of two railway tracks with axle-load values of $Q = 16$ t and $Q' = 22.5$ t, respectively, and assuming that the speed is equal in both cases, the following mathematical equation applies:

$$\frac{C'_o}{C_o} = 1.406^{\beta/\alpha} \tag{16.3}$$

where

C_o, C'_o: the respective maintenance cost

α, β: coefficients that are empirically determined (Esveld, 1989; Lichtberger, 2005), depending on the type of the superstructure wear

The ratio C'_o/C_o is calculated between 1.41 and 2.78 (the ratio β/α is calculated between 1 and 3.5), that is, the maintenance cost in the case of a track for which $Q' = 22.5$ t is between 41% and 178% greater than the respective cost for a track for which $Q = 16$ t.

16.5 ECONOMIC EFFICIENCY OF HEAVY HAUL RAIL TRANSPORT

A question faced by many railway operators nowadays is: 'What is more economically efficient for a railway company? Operating of conventional load freight trains or heavy haul freight trains on a new railway corridor which is dedicated for freight operation?'

The issue of economic viability of rail networks that are used for the heavy haul transport over long distances contains several uncertain factors which affect the cost. The increased axle load initially seems as a profitable factor; however, a more realistic assessment is deemed necessary. The increased costs caused by the increased energy consumption, the increased investment in rolling stock, the requirements for the components of the infrastructure, the more frequent and increased wear of the track and the rolling stock constitute factors that may change the facts.

A technological solution is to increase the maximum allowable payload for a given axle load by improving the net percentage of tare.

Moreover, the use of multiple axle vehicles, which is more popular for road transport, could be a solution for railways as it increases the transport capacity. On the contrary, the strength and safety of bridges should be examined from scratch.

Mathematical models for decision-making regarding the transfer from a conventional load freight system to heavy load system have been developed. In a survey that was carried out recently (Christogiannis, 2012; Pyrgidis and Christogiannis, 2013), the economic efficiency of heavy haul rail transport rail (axle load of 30 t) was compared with the economic efficiency of rail freight transport of conventional loads (axle load 22.5 t) with the aid of mathematical models. The comparison concerns the implementation and operation of a new single track of standard gauge dedicated for freight traffic and is implemented by considering various values of freight workload demand (10,000–130,000 t per day per direction) and of connection lengths (S = 500 and 1,000 km).

Within the framework of this research, the rail infrastructure manager is also the owner of the rolling stock and the operating company. The financial indicator that has been considered to characterise the economic profitability of the new railway corridor is the net present value (NPV) of the investment.

The following steps were methodologically followed:

- Initially, a minimum daily freight load value was considered to be equal to 10,000 t per direction for both corridors. It was considered that this demand
  - As concerns the conventional network, is served by 10 trains per direction which are formed of a number of locomotives and a number of wagons that can meet the above requirement. All wagons have an occupancy ratio of 80%.
  - As concerns the heavy haul corridor, is also served by 10 trains per direction which are formed of a number of locomotives and a number of wagons and can meet the above demands given the same occupancy ratio (80%).

In both operation scenarios, a necessary prerequisite is that, in accordance with the UIC method, the track capacity saturation ratio should not exceed 70%. Assuming a connection length equal to S = 500 and 1,000 km, the NPV is thus estimated for both systems.

- The value of the daily freight load increased by 100% (20,000 t) for both corridors. Thus in order to meet this demand is was considered that
  - As concerns the conventional rail corridor, the number of wagons is initially increased (maximum value of 28 wagons) and thereafter, if demand cannot thus be met, the number of scheduled trains is increased. The occupancy ratio of the wagons remains constant and the track capacity saturation ratio does not exceed 70% of the practical capacity of the line, in accordance with the UIC method. In each case, the number of locomotives required is calculated.
  - As concerns the heavy haul rail corridor, the number of wagons is initially increased (maximum value of 85 wagons) and thereafter, if demand cannot thus be met, the number of scheduled trains is increased. The occupancy ratio of the wagons remains constant and the track capacity saturation ratio does not exceed 70% of the practical capacity of the line. In each case, the number of locomotives required is calculated.

Assuming the connection length to be equal to S = 500 and 1,000 km, the NPV is thus calculated for both corridors

- The value of the daily freight load is gradually increased by steps of 10,000 t, and the same procedure is repeated.
- After being suitably recorded, the results are compared and evaluated.

Table 16.3 indicatively presents, for a connection length of S = 1,000 km and for the different freight volume values under examination, for both exploitation scenarios:

- The formation of the train (locomotive and wagons)
- The number of daily services per direction
- The saturation ratio of track capacity
- The NPV for each of the two scenarios being compared

Table 16.3 Application of the mathematical model – results for connection length S = 1,000 km

	Conventional axle-load line				Heavy axle-load line			
Demand (t/day/direction)	Trains scheduled per day per direction	Train formation (locomotive + wagons)	Track capacity saturation ratio (max permitted = 70%)	NPV (€M)	Trains scheduled per day per direction	Train formation (locomotive + wagons)	Track capacity saturation ratio (max permitted = 70%)	NPV (€M)
10,000	10	1 + 18	29%	–7,205	10	1 + 13	34	–8,986
20,000	13	2 + 28	37%	–2,374	10	1 + 26	34	–4,398
30,000	19	2 + 28	54%	1,976	10	1 + 38	34	–58
40,000	25	2 + 28	71%	6,311	10	1 + 51	34	4,534
50,000	–	–	EXCA	–	10	1 + 63	34	8,785
60,000	–	–	EXCA	–	10	2 + 76	34	13,233
70,000	–	–	EXCA	–	11	2 + 85	38	19,412
80,000	–	–	EXCA	–	12	2 + 85	41	22,461
90,000	–	–	EXCA	–	14	2 + 85	48	28,433
100,000	–	–	EXCA	–	15	2 + 85	52	31,462
110,000	–	–	EXCA	–	17	2 + 85	59	37,447
120,000	–	–	EXCA	–	19	2 + 85	66	43,525
130,000	–	–	EXCA	–	20	2 + 85	69	46,461

Source: Adapted from Pyrgidis, C. and Christogiannis, E. 2013, Freight dedicated railway corridors for conventional and for heavy loads – A comparative assessment of the economic profitability of the two systems, 3rd International Conference on Recent Advances in Railway Engineering – ICRARE 2013, Iran University of Science and Technology, 30 April – 1 May, 2013, Teheran, Congress Proceedings.

It is noted that the initials EXCA (EXceeded CApacity) indicate that 70% of the practical capacity of the track is exceeded and, for this reason, the financial indicator is not recorded.

The diagram in Figure 16.3 shows the change in NPV in relation to freight volume demand for both operation scenarios examined and for both connection lengths considered.

By examining all the combinations of demand and connection length, the following conclusions have been reached:

- The conventional load freight-dedicated corridor can serve up to around 40,000 t daily for each direction, while the heavy haul rail corridor can cater for roughly three times that volume.
- Both systems have a negative NPV for a daily freight of up to approximately 20,000 t for each direction.
- For daily freights per direction of up to 40,000 t, which can be served by both systems, conventional load corridors are economically more profitable.
- For heavy haul rail corridors with a daily freight greater than around 30,000 t, the increase in the connection length results in a significant increase in profitability as it translates to an approximate doubling of the NPV. Similar conclusions also apply for conventional freights; however, the point where it becomes profitable is at around 25,000 t.

The histogram in Figure 16.4 presents the different cost incurred for the two exploitation scenarios examined, for daily freight volumes per direction equal to 30,000 t and for connection length S = 1,000 km.

The intermediate calculations showed that in the case of the heavy haul rail corridor in comparison with the conventional freight corridor:

- The total construction cost of the infrastructure (studies, expropriations, civil engineering works, superstructure, substructure, track systems and facilities) is approximately 18.5% more.

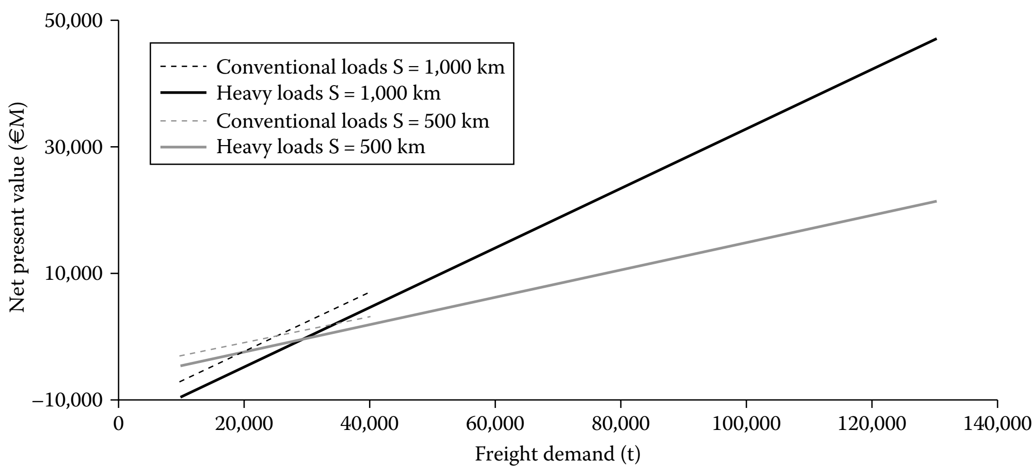

Figure 16.3 Variation of NPV in relation to the freight demand for a conventional axle-load line and for a line for heavy axle loads – length of connection S = 500 and 1,000 km. (Adapted from Pyrgidis, C. and Christogiannis, E. 2013, Freight dedicated railway corridors for conventional and for heavy loads – A comparative assessment of the economic profitability of the two systems, 3rd International Conference on Recent Advances in Railway Engineering – ICRARE 2013, Iran University of Science and Technology, 30 April – 1 May, 2013, Teheran, Congress Proceedings.)

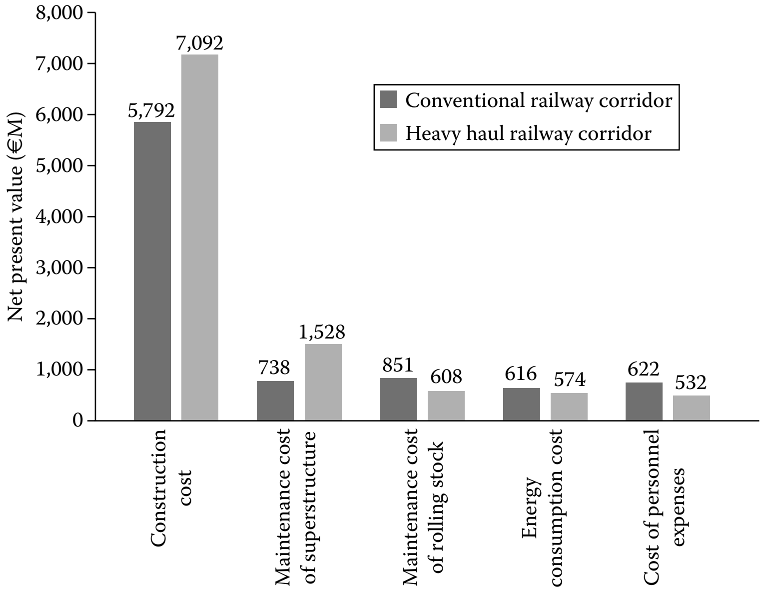

Figure 16.4 Construction, maintenance and operational cost for conventional and heavy axle-load lines – length of connection S = 1,000 km, demand for freight = 30,000 t per day per direction.

- The construction cost of the superstructure is 15% more.
- The maintenance cost of the superstructure is about 52% more.

On the contrary, the cost for the maintenance of the rolling stock, the energy consumption cost and the personnel cost are lower.

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Chapter 17

Impact of traffic composition on the economic profitability of a railway system

17.1 TRAFFIC COMPOSITION AND CLASSIFICATION OF RAILWAY NETWORKS/CORRIDORS

The definition of the term 'traffic composition' was given in Section 1.4.4. On the basis of their traffic, railway networks/corridors were classified into the following five traffic categories:

i. Network/corridor exclusively used by freight trains (freight-dedicated network/corridor)
ii. Network/corridor mainly used by freight trains
iii. Network/corridor with mixed traffic operation
iv. Network/corridor mainly used by passenger trains
v. Network/corridor exclusively used by passenger trains (passenger-dedicated network/corridor)

Five criteria have been identified, namely

Criterion 1: The ratio of the passenger cars or freight wagons in operation to the total rolling stock fleet of a railway network/corridor
Criterion 2: The ratio of the daily passenger traffic load to the respective freight traffic load
Criterion 3: The ratio of the passenger trains to the freight trains circulating in the network/corridor
Criterion 4: The ratio of the train-kilometres travelled per year by passenger trains to the respective train-kilometres travelled by freight trains
Criterion 5: The ratio of the passenger-kilometres of passenger trains to the ton-kilometres of freight trains

Furthermore, it should be noted that the above five criteria do not have all the same impact in determining the traffic composition of a network/corridor and the associated network/corridor classification based on traffic composition. In this context, each criterion was assigned a different weight, considering the degree to which its mathematical expression is related to the operational characteristics affected by the traffic composition (type and quantity of the transport volume carried over the network/corridor, track capacity of the railway infrastructure, required amount of track maintenance work, length of the available railway infrastructure and the degree of track capacity occupancy by passenger and freight trains).

Table 17.1 Limits and weighting factors for the criteria proposed for the classification of the railway network sample into the five categories of traffic composition

Categories of traffic composition	*Criterion 1 weighting factor 0.10*	*Criterion 2 weighting factor 0.20*	*Criterion 3 weighting factor 0.27*	*Criterion 4 weighting factor 0.23*	*Criterion 5 weighting factor 0.20*
I	0–0.02	0–0.1	0–0.25	0–0.1	0–0.01
II	0.02–0.15	0.1–1	0.25–5	0.1–1	0.01–0.2
III	0.15–0.25	1–3	5–30	1–4	0.2–2
IV	0.25–2.0	3–50	30–100	4–50	2–5
V	>2	>50	>100	>50	>5

Source: Adapted from Christogiannis, E. and Pyrgidis, C. 2013, An investigation into the relationship between the traffic composition of a railway network and its economic profitability, *Rail Engineering International*, 1, 13–16; Christogiannis, E. 2012, Investigation of the impact of traffic composition on the economic profitability of a railway corridor – Fundamental principles and mathematical simulation for the selection of operational scenario for a railway corridor, PhD Thesis, Aristotle University of Thessaloniki, Thessaloniki, Greece.

Table 17.1 presents for each criterion the limits proposed for classifying the networks into the five categories of traffic composition, as defined above, and the weighting factor of each criterion.

Table 17.2 presents the ranking in the categories of traffic composition of a sample of 20 networks selected (based on published data of the specific railway networks, 2007 data), in accordance with the findings using the above methodology.

In this way, China is classified under traffic composition category II, based on criteria 1, 2 and 3, and under category III, based on criteria 4 and 5. Therefore, China will belong to either category II or category III. On the basis of weighting factors, category II dominates since it is rated at 0.57 ($0.10 + 0.20 + 0.27 = 0.57$), outranking category III rated at 0.43 ($0.23 + 0.20 = 0.43$).

On the basis of 2007 data the conclusions drawn from the final classification of the sample networks are the following:

- Most EU networks are classified under categories III and IV, the majority being in category IV, that is, passenger transportation is predominant.
- The United States, Canada and Lithuania are classified under category I, that is, they are dedicated freight train networks.
- Japan's network is the only category V network, that is, dedicated passenger train network.
- Almost all east European networks are classified under categories II and III, with the majority in category II, that is, emphasis on freight transport.
- South Africa's network is ranked in category II, though on the basis of the weighting factors, category III dominates (rated at 0.47). The reason for this is that category II (network mainly used by freight trains) and category I (dedicated freight train network) together collect a higher ranking ($0.43 + 0.10 = 0.53$) than category III (0.47).

17.2 ECONOMIC PROFITABILITY AND CLASSIFICATION OF RAILWAY NETWORKS/CORRIDORS

As 'economic profitability' of a railway network/corridor is defined as its capacity to produce significant transport volume, compared with other competitive transportation means and to be financially robust, that is, to show profit rather than deficits and debts.

Table 17.2 Results of multi-criteria ranking of the networks from the sample based on traffic composition (data 2007)

		Category I					*Category II*					*Category III*					*Category IV*					*Category V*					
		Criteria					*Criteria*					*Criteria*					*Criteria*					*Criteria*					*Multi-criteria*
Number	*Country*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*ranking*
1	United States	0.10	0.20	0.27	0.23	0.20																					I
Ranking				1.00					–										–					–			
2	Canada	0.10	0.20	0.27	0.23	0.20																					I
Ranking				1.00					–										–					–			
3	Lithuania		0.20				0.10		0.27	0.23	0.20																II
Ranking				0.20					0.80										–					–			
4	Australia						0.10	0.20	0.27	0.23	0.20																II
Ranking				0.00					1.00										–					–			
5	Estonia		0.20				0.10		0.27	0.23	0.20																II
Ranking				0.20					0.80										–					–			
6	Latvia						0.10	0.20	0.27	0.23	0.20																II
Ranking				–					1.00										–					–			
7	Russia						0.10	0.20	0.27	0.23	0.20																II
Ranking				–					1.00										–					–			
8	Ukraine						0.10	0.20						0.27	0.23	0.20											III
Ranking				–					0.30					0.70					–					–			
9	China						0.10	0.20	0.27						0.23	0.20											II
Ranking				–					0.57					0.43					–					–			
10	South Africa	0.10								0.23	0.20		0.20	0.27													II
Ranking				0.10					0.43					0.47					–					–			
11	Poland						0.10	0.20						0.27	0.23	0.20											III
Ranking				–					0.30					0.70					–					–			
12	Austria											0.10	0.20	0.27	0.23	0.20											III
Ranking				–					–					1.00					–					–			

(*Continued*)

Table 17.2 (Continued) Results of multi-criteria ranking of the networks from the sample based on traffic composition (data 2007)

		Category I					*Category II*					*Category III*					*Category IV*					*Category V*					
		Criteria					*Criteria*					*Criteria*					*Criteria*					*Criteria*					*Multi-criteria*
Number	*Country*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*1*	*2*	*3*	*4*	*5*	*ranking*
13	Romania						0.10	0.20						0.27	0.23	0.20											III
Ranking				–					0.30					0.70					–					–			
14	Germany											0.10				0.20		0.20	0.27	0.23							IV
Ranking				–					–					0.30					0.70					–			
15	Spain															0.20		0.20	0.20	0.23		0.10		0.27			IV
Ranking				–					–					0.20					0.63					0.37			
16	India											0.10				0.20		0.20	0.27	0.23							IV
Ranking				–					–					0.30					0.70					–			
17	France															0.20	1.00	0.20	0.27	0.23							IV
Ranking				–					–					0.20					0.80					–			
18	Italy											0.10						0.20	0.27	0.23	0.20						IV
Ranking				–					–					0.10					0.90					–			
19	United Kingdom																1.00	0.20	0.27	0.23	0.20						IV
Ranking				–					–					–					1.00					–			
20	Japan																					0.10	0.20	0.27	0.23	0.20	V
Ranking				–					–					–					–					1.00			

Source: Adapted from Christogiannis, E. and Pyrgidis, C. 2013, An investigation into the relationship between the traffic composition of a railway network and its economic profitability, *Rail Engineering International*, 1, 13–16; Christogiannis, E. 2012, Investigation of the impact of traffic composition on the economic profitability of a railway corridor – Fundamental principles and mathematical simulation for the selection of operational scenario for a railway corridor, PhD Thesis, Aristotle University of Thessaloniki, Thessaloniki, Greece.

The economic profitability of a railway system can be characterised by the following factors:

- The company's profits and debts owed on a fixed time basis (e.g., over a 5-year period)
- The company's position in the competitive market
- The organisation and structure of the company
- The company's growth activities

On the basis of their economic profitability, railway systems (networks) can be classified into the following five categories:

A+: Very positive
A: Positive
AB: Neutral – balanced
B: Negative
B–: Very negative

The methodology for the classification of railway networks and railway corridors in the aforementioned categories according to their economic profitability is outlined hereunder (Christogiannis, 2012; Christogiannis and Pyrgidis, 2013).

Five criteria are used:

Criterion 1: Profit/loss estimation for the railway sector for the last 5-year period
Criterion 2: Average productivity (train-kilometres/number of personnel) over the last 5-year period
Criterion 3: Average market share over the last 5-year period
Criterion 4: Percentage of change in track length of each network over the last 10-year period
Criterion 5: Percentage of change in the number of rolling stock over the last 10-year period

Furthermore, it is considered that the above five criteria do not have all the same impact on the determination of the economic profitability of a network and the associated network classification based on economic profitability. In this context, each criterion was assigned a different weighting, considering the degree to which its mathematical expression is related to the scale of their economic profitability (company profits and amounts owed on a fixed time basis, position of the company in the competitive market, its organisation and structure and its growth activities).

Table 17.3 presents, for each criterion, the classification limits for the five categories of economic profitability defined above and the weighting factor of each criterion.

In calculating the grade, an adjustment factor has been entered in each criterion which takes into account any state intervention in the railway network.

Table 17.4 presents the proposed adjustment in the evaluation of the railway networks on the basis of their economic profitability.

For example, where a railway network may be initially placed in the economic profitability category A+, this might be reduced by one mark (to A) if there exist conditions of strong state protectionism.

17.3 THE PROBLEM OF MIXED TRAFFIC OPERATION

Throughout the world, the vast majority of railway networks/corridors concern mixed train operation (Batisse, 1994, 1995a,b; UIC, 2007). Express and local passenger trains and freight

Table 17.3 Criteria limits and weightings for the classification of the railway networks in the sample for the five categories of economic profitability

Economic profitability category	*Criterion 1 weighting factor 0.35 (M)*	*Criterion 2 weighting factor 0.17*	*Criterion 3 weighting factor 0.20 (%)*	*Criterion 4 weighting factor 0.14 (%)*	*Criterion 5 weighting factor 0.14 (%)*
A+	>€1,500	>4.0	>25	>5	>15
A	€100 to €1,500	3.0–4.0	20–25	0–5	–5 to 15
AB	€100 to –€60	2.0–3.0	15–20	–5 to 0	–5 to –15
B	–€60 to –€100	1.0 –2.0	10–15	–10 to –5	–15 to –30
B–	>–€100	<1.0	<10	>–10	>–30

Source: Adapted from Christogiannis, E. and Pyrgidis, C. 2013, An investigation into the relationship between the traffic composition of a railway network and its economic profitability, *Rail Engineering International*, 1, 13–16; Christogiannis, E. 2012, Investigation of the impact of traffic composition on the economic profitability of a railway corridor – Fundamental principles and mathematical simulation for the selection of operational scenario for a railway corridor, PhD Thesis, Aristotle University of Thessaloniki, Thessaloniki, Greece.

trains are routed on the same track. For many years, this was the basic rule in the rail transport sector. On the one hand, this practice seems to achieve economies of scale, as most trains use the same railway infrastructure; on the other hand, it creates problems in the operation and maintenance of the network, as trains of different functionality circulate on the same track. More specifically, many features of the freight wagons/trains differ substantially from those of the passenger cars/trains. As a result, sharing the same track affects the design, construction, operation and maintenance of a railway system either directly or indirectly.

Table 17.5 shows the qualitative impact of the features which differ substantially between freight and passenger vehicles/trains, on the components of a railway system.

A question which is a matter of concern to many railway companies is

> What is more economically profitable for a railway company? The simultaneous routing of passenger and freight trains (mixed operation) on a railway network/corridor or the differentiation between passenger and freight traffic (dedicated operation)? Which are the criteria and basic principles that will lead railway operators to select the operational framework that they will adopt
>
> For a new railway connection which they are planning to construct?
> For an existing railway connection of mixed traffic operation where demand is modified?

Mixed traffic operation networks satisfy primarily passenger transportation. This priority usually leads to resource inadequacy for freight trains, which are further delayed in favour of passenger trains. The needs of the freight transportation market differ from those of passenger transportation. This situation seems to enforce the gradual segregation of railway networks/corridors for passenger and freight transportation. Indicatively, it should be noted that

Table 17.4 Proposed adjustment (change in category of economic profitability depending on extent of state intervention)

Category of state intervention	*Proposed adjustment correction in the category of economic profitability*
a Fully competitive	One-mark increase
b Partially competitive	Unchanged
c State protectionism	One-mark decrease

Table 17.5 Qualitative impact of train features on the components of a railway system – requirements

Train/vehicle feature – variation	*Effects on the railway system's components – requirements*
Maximum running speed – increase Is greater for passenger trains (120–320 [350] km/h) than for freight trains (60–120 km/h)	*Increase*: Track design dynamic load, train braking distance, centrifugal force in curves, aerodynamic train resistance, track capacity, and consequences in case of accident *Requirements*: Larger distance between track centres, higher curvature radius in the longitudinal and vertical alignment, higher track cant, greater length of transition curves in the horizontal alignment, continuously welded and heavier rails, concrete sleepers, elastic fastenings, thicker track bed layers, track fencing, electric signalling, longer signal spacing, electrification (for $V > 160$–180 km/h), specific rolling stock, slow train overtaking, increased safety measures along the track, bigger tunnel useful cross section, and higher maintenance needs.
Axle load – increase Is lower for passenger trains (12–18 t) than for freight trains (conventional loads: 16–25 t)	*Increase*: Track design static load, train braking distance, track geometry defects deterioration rate, and train movement resistance *Requirements*: Less steep gradients, heavier rails, thicker track bed layers, longer signal spacing, greater traction power requirements, and higher maintenance needs
Train weight – increase Is much smaller for passenger trains than for freight trains	*Increase*: Braking weight and train movement resistance *Requirements*: Less steep gradients, longer signal spacing, and greater traction power requirements
Train length – increase Is much smaller for passenger trains	*Decrease*: Track capacity *Requirements*: Longer tracks and platforms in stations
Daily traffic load – increase	*Increase*: Track maintenance needs and track geometry defects deterioration rate *Requirements*: Heavier rails, thicker track bed layers, and higher maintenance needs
Dynamic passenger comfort Concerns mainly the passenger cars	*Requirements*: In the case of passenger trains, lateral residual centrifugal acceleration <1.0 m/s^2, greater curvature radius in the longitudinal and vertical alignment, higher track cant, and greater length of transition curves in the horizontal alignment In the case of mixed traffic operation, adoption of track cant which satisfies both train categories
Punctuality	*Requirements*: Imperative for passenger trains – desirable for freight trains
Vehicle clearance gauge	*Requirements*: Differentiates depot and station dimensioning, larger distance between track centres and civil engineering structures height clearance
Terminal stations	*Requirements*: Totally different design and equipment for passenger and for freight stations. Passenger trains stop much more frequently (every 50–100 km) than freight trains (every 300–800 km)
Transported goods	*Requirements*: Different safety measures required for the infrastructure and the rolling stock when transporting passengers compared to cargo. Special safety measures in tunnels when carrying passengers and special safety measures in the 'open' line when carrying dangerous goods

Source: Adapted from Christogiannis, E. and Pyrgidis, C. 2013, An investigation into the relationship between the traffic composition of a railway network and its economic profitability, *Rail Engineering International*, 1, 13–16; Pyrgidis, C. and Christogiannis, E. 2011, The problem of the presence of passenger and freight trains in the same track and their impact on the profitability of the railways companies, *9th World Congress on Railway Research 'Meeting the challenges for future mobility'*, 22–26 May 2011, Lille, France, Congress Proceedings (CD); Pyrgidis, C. and Christogiannis, E. 2012, The problem of the presence of passenger and freight trains on the same track, *Elsevier Procedia Social and Behavioral Sciences*, 48, 1143–1164.

- Large railway networks, particularly those of countries with significant industrial output, such as in China, Russia, India, etc., have constructed or are planning to construct dedicated passenger and/or freight railway corridors (OECD, 2002; ADB (Asian Development Bank), 2005; Woodburn et al., 2008; Dedicated Freight Corridor Corporation of India, 2015).
- The construction of dedicated passenger railway corridors in Japan has proven to be an especially successful and financially profitable investment, and, as such, the Japanese railway network is growing continuously, and railway has secured a large part of the market.
- The abandonment of conventional long-distance passenger services in the United States and the shift by railway companies toward freight transportation has resulted in significant profits for these companies and has significantly increased the share of the railway in freight transportation.
- The trend toward a global economy without restrictions in freight movement is prompting large networks worldwide to design and create international dedicated railway freight corridors.
- The World Bank recommends that railway organisations in distress should proceed to managerial separation of passenger and freight transportation activities.
- The competent Directorates of Transport of the European Union seek to establish criteria and basic principles by dint of which choices will be made in regard to the operational framework which will be adopted for upgraded or newly designed lines.

An essential prerequisite for the creation of long dedicated freight railway corridors would be the existence of connection amongst the networks that will be served and the ensuring of interoperability.

17.4 INVESTIGATION OF THE IMPACT OF TRAFFIC COMPOSITION ON THE ECONOMIC PROFITABILITY OF A RAILWAY SYSTEM

17.4.1 Data published by the railway networks

In a survey conducted in 2007, based on data publicly available from the networks themselves, traffic composition was associated with the profitability of a railway network (Christogiannis and Pyrgidis, 2013).

In particular the 20 networks listed in Table 17.2 were thoroughly examined.

Figure 17.1 illustrates the results of this procedure. The following conclusions may be deduced from this figure:

- Networks belonging to traffic composition categories I and V, that is, networks with dedicated freight and passenger operation, respectively, belong to the category of economic profitability A+, that is, they present very positive economic profitability.
- The majority of networks belonging to the traffic composition category III, that is, with mixed traffic operation, present a negative or very negative economic profitability.
- The majority of networks with emphasis on freight train traffic show a positive balanced economic efficiency while no network shows a very negative profitability.
- Networks with emphasis on passenger train traffic, by majority, do not show a negative profitability.

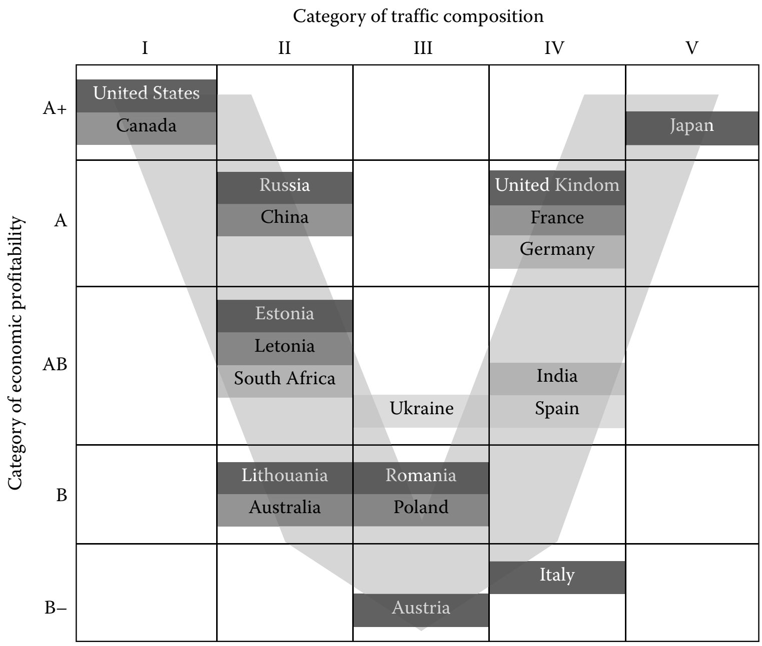

Figure 17.1 Correlation between traffic composition and economic profitability of a railway network. (Adapted from Christogiannis, E. and Pyrgidis, C. 2013, An investigation into the relationship between the traffic composition of a railway network and its economic profitability, *Rail Engineering International*, 1, 13–16.)

As can be seen in the translucent grey 'V' in Figure 17.1, the more dedicated (passenger or freight) the traffic composition is, the more the increase in the economic profitability of networks. On the contrary, the more mixed the traffic composition tends to be, the more the decline in their economic profitability.

17.4.2 Mathematical simulation

17.4.2.1 Selection of the operational framework for a new railway corridor

The impact of traffic composition on the economic profitability of *a new railway corridor* has been investigated with the aid of a mathematical model as part of a PhD dissertation thesis (Christogiannis, 2012).

Within the framework of this research, the rail infrastructure manager is also the owner of the rolling stock and the operating company. Two financial indicators were considered to be the figures that characterise the economic profitability of a new railway corridor: (a) the NPV of the investment and (b) the internal rate of return (IRR) of the investment.

The five operation scenarios of a new railroad connection presented in Table 17.6 were examined with the help of a model (Pyrgidis and Christogiannis, 2013a; Christogiannis and Pyrgidis, 2014, 2015).

The mathematical model that was developed allows, for various operation scenarios concerning the traffic composition, the following:

- Calculation of the expenses which are necessary for the implementation of the new railway link. The expenses calculated by the model for each of the five operation scenarios examined include the cost for the construction of the new track infrastructure

Table 17.6 New railway connection – Operation scenarios under study – Basic railway system design parameters

Schematic image of the scenario	*Description*	*Basic railway system design parameters*
Freight and passengers	Scenario S1: double track – mixed traffic operation	V_d = 200 km/h, Q_{max} = 22.5 t, i_{max} = 2%, V_{pas} = 200 km/h, V_{fr} = 120 km/h, rails UIC 60, and γ_{ncmax} = 0.50 m/s^2
Passengers	Scenario S2: double track – passenger-dedicated operation	V_d = 200 km/h, Q_{max} = 22.5 t, i_{max} = 2.5%, V_{pas} = 200 km/h, rails UIC 60, and γ_{nmaxc} = 0.50 m/s^2
Freight	Scenario S3: double track – freight-dedicated operation	V_d = 120 km/h, Q_{max} = 25 t, i_{max} = 1.5%, V_{fr} = 120 km/h, rails UIC 70, and γ_{ncmax} = 1.0 m/s^2
Freight	Scenario S4: single track – freight-dedicated operation	V_d = 120 km/h, Q_{max} = 25 t, i_{max} = 1.5%, V_{fr} = 120 km/h, rails UIC 70, and γ_{ncmax} = 1.0 m/s^2
Freight and passengers	Scenario S5: single track – mixed traffic operation	V_d = 200 km/h, Q_{max} = 22.5 t, i_{max} = 2%, V_{pas} = 200 km/h, V_{fr} = 120 km/h, rails UIC 60, and γ_{ncmax} = 0.50 m/s^2

Source: Adapted from Christogiannis, E. 2012, Investigation of the impact of traffic composition on the economic profitability of a railway corridor – Fundamental principles and mathematical simulation for the selection of operational scenario for a railway corridor, PhD Thesis, Aristotle University of Thessaloniki, Thessaloniki, Greece.

V_d, track design speed; V_{pas}, V_{fr}, maximum speed of passenger and freight trains, respectively; Q_{max}, maximum axle load or vertical design axle load of a railway infrastructure; i_{max}, maximum longitudinal gradient and γ_{ncmax}, maximum permitted residual centrifugal acceleration.

(substructure, civil engineering works, superstructure, stations, electrification and signalling systems, level crossings, expropriations, rolling stock maintenance facilities, and studies), the cost for rolling stock acquisition, the operating costs, the maintenance cost and the financial cost.

- Calculation of the revenue generated for the undertaking. Revenue is considered to originate from passengers and freight, while the residual value of the project and revenue from loans are also included in the calculation.
- Calculation of the economic profitability of each exploitation scenario on the basis of the method of the NPV and IRR of the investment, for the duration of the financial lifespan of the investment.
- Comparison of the operation scenarios (reference scenarios) regarding their economic profitability in general and, more specifically, regarding their individual cost elements and other railway system parameters (e.g., infrastructure cost and maintenance cost).
- Studying of the influence of various design, construction, operational and financial parameters of the railway system on the system's economic profitability.
- Selection, on the basis of demand for passenger and/or freight volume to be transported via rail on a connection, of the operation scenario that allows the highest economic profitability.

The conclusions drawn are grouped into the following three 'areas':

1. *Comparison of economic profitability scenarios*
    - Among scenarios S1, S2 and S3, which concern the construction and operation of a new railway corridor consisting of a double track, the most economically advantageous solution appears to be scenario S3, that is, the case of dedicated freight train operation, followed by scenario S2, that is, dedicated passenger train operation, followed by mixed train traffic operation.

- Between scenarios S4 and S5, which concern the construction and operation of a new railway corridor consisting of a single track, the most economically advantageous solution appears to be scenario S4, that is, the case of dedicated freight train traffic. In fact, scenario S4 is almost equivalent to scenario S1.

2. *Influence of various railway system parameters on economic profitability*
   - As the length of the railway connection increases, the NPV of the investment and, thus, the economic profitability of the railway system also increase. Scenarios S3, S2 and S1 have an NPV equal to €11,000 M for connection lengths of S = 580, 760 and 1,120 km, respectively.
   - The economic profitability of all operation scenarios decreases drastically as the topography of the landscape becomes harsher. In fact, for mountainous topography, it is negative in all cases. The change in topography of the landscape from average evenness to flatlands results in a significant increase in the NPV. More specifically, this increase is equal to 89% for scenario S1, 50% for scenario S2, 46% for scenario S3, 69% for scenario S4 and 190% for scenario S5.
   - The change in the composition of traffic significantly affects the economic profitability of scenarios S1 and S5 (Figure 17.2). In the case of scenario S1, the curve of the diagram of the ratio of passenger trains to the total number of routed trains and of the NPV has a parabolic shape which points downwards. This curve reaches its minimum in the case of mixed traffic (50% passenger–50% freight trains). The maximum values of the NPV in scenario S1 appear in the case of dedicated train operation and are 32% higher in the case of dedicated freight train operation in relation to the case of dedicated passenger train operation.
3. *Selection criteria of rail corridor operational framework*
   The basic criterion for the selection of an operation scenario for a new railway corridor concerns the characteristics of transportation demand and, particularly, the type of goods that are being transported in combination with the volume being transported and the topography of the landscape.

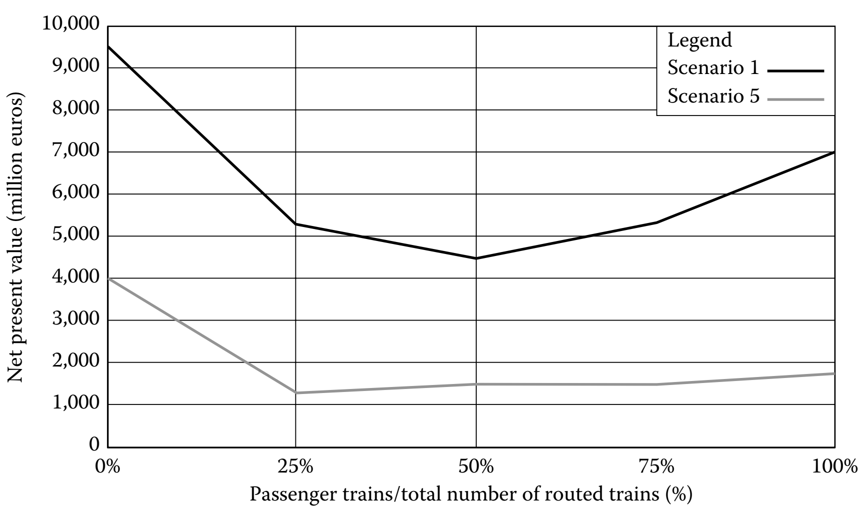

Figure 17.2 Variation of the NPV in relation to the traffic composition – scenarios S1 (double track) and S5 (single track). (Adapted from Christogiannis, E. and Pyrgidis, C. 2014, Investigation of the impact of traffic composition on the economic profitability of a new railway corridor, *Journal of Rail and Rapid Transit,* Proceedings of the IMechE, Part F, 228(4), 389–401.)

- For average demand values (e.g., 9,000 passengers per day per direction/81,000 t per day per direction, 25,000 pas/25,000 t), scenario S1 (double track – mixed operation) presents the highest NPV.
- For high demand values (e.g., 18,000 passengers per day per direction/162,000 t, 50,000 pas/50,000 t), dedicated traffic scenarios (i.e. 0 pas/162,000 t, 50,000 pas/0 t) presents the highest NPV, both in the case of a single track and in the case of a double track. Furthermore, S3, the dedicated freight operation scenario presents a higher NPV in relation to S2, the dedicated passenger operation scenario.
- When the demand ratio of the number of passengers to the freight tonnes approaches 1/3 (traffic composition percentage 50%–50%), this 'favours' mixed operation scenarios (scenarios S1 and S5).

By comparing the results deriving from the two approaches (published railway networks data and mathematical simulation), compatible conclusions are drawn. In particular, the curves of Figures 17.1 and 17.2 have exactly the same form and lead, at least qualitatively, to the same conclusions regarding the impact of the traffic composition on the economic profitability of the railway system.

17.4.2.2 Selection of the operating framework for an existing railway corridor

Pyrgidis and Christogiannis (2013b) and Christogiannis and Pyrgidis (2015) show the conditions under which it is economically viable to change the operating framework of an existing railway corridor from mixed traffic to dedicated traffic.

As part of this research, the railway infrastructure manager is both the owner of the rolling stock and the operator. Two economic indicators were considered as the main features that characterise the economic profitability of a railway corridor, namely (a) the NPV of the investment and (b) the IRR of the investment.

The three operating scenarios that are described in Table 17.7 were examined and compared. The existing rail link which is currently in operation is implemented with a single track of standard gauge, where traffic is allowed in both directions.

For the mathematical simulation of the three scenarios of Table 17.7, the steps outlined hereunder are followed:

Regarding the current situation (*scenario S1*)

- Maintenance and operation cost for the existing rail corridor is calculated.
- Revenue from the operation of the existing system is calculated, given the demand of traffic volumes and the fares.
- The NPV of the existing rail corridor for a time period $t_{fin} - t'$ (t_{fin}: year of the end of the corridor's economic life and t′: year of change of the corridor's operating frame).

In the case of scenario S1 (mixed traffic operation), the ratio of the number of routed passenger trains to routed freight trains is 3:1 (75% of passenger trains and 25% freight trains). The saturation rate of the line's capacity is already equal to 70% and may not be further increased. On this basis the transport volume, for both passenger and freight, is specified.

The line is considered to be saturated and it cannot respond to new increased demand.

Table 17.7 Existing railway infrastructure – Operation scenarios under consideration – Basic railway system design parameters

Schematic image of the scenario	*Description*	*Basic railway system design parameters*
Passengers and freight	Scenario S1: Current situation; single track – mixed traffic operation	V_d = 160 km/h, Q_{max} = 22.5 t, i_{max} = 20‰, V_{pas} = 160 km/h, V_{fr} = 100 km/h, electrification, and electric side signalling
Passengers	Scenario S2: New situation: single track – passenger-dedicated operation	V_d = 160 km/h, Q_{max} = 22.5 t, i_{max} = 20‰, V_{pas} = 160 km/h, electrification, and electric side signalling
Freight	Scenario S3: New situation: single track – freight-dedicated operation	V_d = 160 km/h, Q_{max} = 22.5 t, i_{max} = 20‰, V_{fr} = 120 km/h, electrification, and electric side signalling

Source: Adapted from Pyrgidis, C. and Christogiannis, E. 2013b, Mixed or pure train routing? The case of an existing railway corridor, *6th International Conference for Transport Research ICTR*, 17–18 October 2013, Thessaloniki, Conference Proceedings; Christogiannis, E. and Pyrgidis, C. 2015, Selection of the optimum exploitation scenario for an interurban railway corridor by the help of mathematical models, *Ingegneria Ferroviaria*, 5, 427–448.

Regarding the new situation (*S2 and S3 scenarios*)

- For both considered scenarios of dedicated operation, the required changes to the existing infrastructure (replacement of track superstructure components, changes in the signalling plan, upgrading of railway stations, etc.) are recorded.
- For the new scenarios of dedicated operation, it is considered that the demand for passenger or freight transport initially remains the same in comparison with the existing situation and then it gradually increases by a specific percentage.
- The required changes in rolling stock in order to serve the new demand (additional rolling stock) are recorded.
- The required changes in the operating system (more scheduled trains daily, additional stations of different category, changes in the number of required staff, etc.).
- The cost of implementing the above required interventions, the maintenance cost and the operating cost of the new dedicated operation railway corridor are calculated.
- The revenue (for a given fare price) from the operation of the new system is calculated.
- The NPV of the railway corridor for a period $t_{fin} - t'$ is calculated.
- The NPVs of scenarios S2 and S3 are compared with the respective NPV of scenario S1 and with each other and the limit of the demand for which the dedicated operation scenarios are more cost-effective than the mixed operation on one is sought.

For both dedicated traffic scenarios and for the case of a new situation, it is considered that the line capacity saturation percentage may not exceed 70% while the train formation remains stable, which means that additional demand is treated by adding and routing extra trains. On this basis, for the new operating scenarios, the increased demand for passenger or freight transport is catered for by increasing the scheduled trains.

For scenario S2 (dedicated passenger operation), it is considered that the freight transport volume that will no longer be carried by rail will be carried by other modes of transport. The same applies for the passenger transport volume in the case of scenario S3 (dedicated freight operation).

The conclusions drawn from the above analysis are summarised as follows:

Scenario S1 (current situation – mixed operation) presents a low profit for a connection length S = 500 km. Profitability is approximately tripled when the connection length is equal to S = 1,000 km. The 70% saturation rate responds to the routing of 19 trains per day per direction at a ratio of passenger to freight trains that is equal to 3:1.

A comparison between the economic profitability of scenario S2 (dedicated passenger operation) and the economic profitability of scenario S1 (existing mixed operation) shows that

- For a rail connection which has a length of S = 500 km, scenario S2 is more efficient than scenario S1 if the passenger volume is nearly doubled (i.e. if it is increased from 8,000 to 15,000 passengers per direction per day).
- For a rail connection which has a length of S = 1,000 km, scenario S2 is more efficient than scenario S1 if the passenger volume is increased by 20% or more (i.e. if it is increased from 8,000 to 9,600 passengers per direction per day).

A comparison between the economic profitability of scenario S3 (dedicated freight operation) and the economic profitability of scenario S1 (existing mixed operation) shows that

- For a rail connection with a length of S = 500 km, scenario S3 is more efficient than scenario S1 if the passenger volume is nearly tripled (i.e. if it is increased from 11,000 to 30,000 tons per direction per day).
- For a rail connection with a length of S = 1,000 km, scenario S3 is more efficient than scenario S1 if the passenger volume is doubled (i.e. if it is increased from 11,000 to 30,000 tons per direction per day).

A comparison between the economic profitability of scenario S2 (dedicated passenger operation) and the economic profitability of scenario S3 (dedicated freight operation) shows that

- For both rail connection lengths that are being considered and for a saturation rate of the line capacity that is greater than 40%, scenario S3 is more efficient than scenario S2.
- For a saturation rate of the line capacity that is equal to 68%, scenario S3 is more efficient than scenario S2 by approximately 19% for a rail connection which has a length of S = 500 km and by 7% for a rail connection which has a length of S = 1,000 km.

Regardless of the exploitation scenario, as the length of the rail connection increases, the NPV of investment and, therefore, the economic profitability of the railway system also increase. The increase rate for NPV in relation with the rail connection length is greater for scenario S3, followed by the respective rates for scenarios S2 and S1.

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Chapter 18

Railway safety

18.1 TYPES OF RAILWAY INCIDENTS AND DEFINITION OF RAILWAY SAFETY

18.1.1 Types of railway incidents

Railway incidents are classified into three groups: accidents, events and failures. Relevant definitions are given below (Pyrgidis and Kotoulas, 2006; ERA, 2009)

- *Accident*: All undesired or unexpected sudden incidents or a specific chain of similar incidents that have (or had) undesirable consequences for the railway system (railway infrastructure, rolling stock, and railway operation) and the environment.
- *Event*: Every incident not characterised as an accident that concerns the operation of the trains and also that affects their safety. An event may be the cause of an accident.
- *Failure*: Failure represents a specific category of an event. It can be defined as any technical malfunction of the railway infrastructure and of the rolling stock that affects the safety of the operation of the whole system. Accordingly, a failure may be the cause of an accident.
- *Incident:* It is a unified definition of an accident, an event or a failure.

18.1.2 Definition of railway safety

The safety that a railway system provides to its users can be defined with the aid of the two following approaches.

18.1.2.1 Risk level

This approach suggests a qualitative assessment of safety. In the case of a railway system the term 'safety' describes the guarantee, through the constituents and the components of the railway system, that during operation the risk level is not described as 'non-permissible' (European Standard EN50126-1, 2000; EC, 2009).

The classification of the risk is uniquely accrued by the combination of the frequency and the severity of an event. This correlation defines the following four risk levels (Table 18.1):

- *Non-permissible*: Accidents of this category must be eliminated. It represents the most significant category and necessitates urgent safety measures by the services responsible, regardless of the financial and operational cost.
- *Non-desirable*: Accidents of this category can be accepted only in case of inability to contain their consequences and always upon the relevant approval of the authority in charge.

Table 18.1 Risk levels based on the frequency and severity of accidents

		Accident severity			
	Risk levels	*Catastrophic*	*Severe*	*Minor importance*	*Negligible*
Accident frequency	Frequent	Non-permissible	Non-permissible	Non-permissible	Non-desirable
	Possible	Non-permissible	Non-permissible	Non-desirable	Permissible
	Occasional	Non-permissible	Non-desirable	Non-desirable	Permissible
	Unusual	Non-desirable	Non-desirable	Permissible	Unimportant
	Rare	Permissible	Permissible	Unimportant	Unimportant
	Unlikely	Unimportant	Unimportant	Unimportant	Unimportant

Source: Adapted from European Standard EN50126-1 2000, *Railway Applications: Reliability, Availability, Maintainability and Safety* (*RAMS*), Part 1, CENELEC European Standards (European Committee for Electromechanical Standardization).

- *Permissible*: It corresponds to a generally acceptable safety level, without excluding further improvements, if it is feasible.
- *Unimportant*: The incidents of this category are acceptable, provided that there is approval of the competent authority.

Regarding the classification of accidents according to the severity of their consequences, various attempts have been made in Europe. The EU has introduced a common ground for these efforts. More specifically, the following definitions are suggested:

- *Catastrophic*: Fatalities and/or multiple severe injuries and/or severe environmental impact and/or extensive property damage.
- *Severe*: One fatality and/or serious injury, and/or significant environmental impact, and/or limited severe property damage.
- *Low severity*: Minor injury, and/or significant threat (or low impact) on the environment, and/or limited damage.
- *Negligible*: Possible minor injury and/or minor property damage.

To the classification stated above, an additional definition could be introduced, and more specifically 'events and failures that did not lead up to an accident'.

The EU by adopting standards and by forming the appropriate legislative framework has quantified some of the consequences mentioned above. For example, as extensive damages are considered 'those for which the Accident Investigation Body can directly estimate that a minimum of €2,000,000 are required for their restoration' (Directive 2004/49).

However, there is no precise definition of the consequences and therefore the above classification is applied at will by initiatives of railway safety stakeholders. Thus, according to the British Railtrack, the equivalence between fatalities and injuries is defined as follows:

- 1 fatality = 10 severe injuries
- 1 severe injury = 20 minor injuries

On the contrary, as far as incident frequency is concerned, there are as of yet no standards clearly defining the borderlines between the various classifications (possible, circumstantial, etc.), and this renders the application of Table 18.1 difficult.

In Section 18.9, a methodology for the classification of accident frequency is suggested.

18.1.2.2 Incident 'indicators'

This approach suggests a quantitative assessment of safety. The safety that a railway system provides is evaluated by the incidents that occurred during a specific time period (e.g., 1 year) and had consequences on the track, the rolling stock, the passengers, the cargo and the environment. In this context, indicators based on incidents that have occurred over a given time period are being used (ERA, 2013).

Worldwide various countries use specific indicators to assess the safety of their railway networks. These indicators differ very little among the various countries (FRA, 2003; Australian Government, 2012; Indian Government, 2013; Japan Transport Safety Board, 2013).

To apply the EU Directive 2004/49 for safety and its revision 2009/149/EC (EC, 2009) ERA (European Railway Agency) proposed a series of indicators (ERA, 2013) concerning rail incidents, their impact in relation to human life, economic impact, technical impact, etc. More emphasis is placed on human life, as any incident is directly related to its consequences upon it. Consequences may include fatality, serious injury and light injury. These consequences combined with the number of accidents and the economic impact form the values of corresponding indicators. These indicators are necessary for further decisions on the prevention measures that need to be adopted.

These indicators are as follows:

Indicators related to accidents (*per year*)

- Total number of serious accidents (number)
- Relative number of serious accidents (number/train-kilometre)
- Distribution of accidents per accident category
- Fatality risk indicator: death toll as a result of train accidents per million train-kilometre

Taking into consideration all fatalities from rail accidents (excluding suicides), the EU 'fatality risk indicator' in 2009–2011 had a value of 0.31 fatalities per million train-kilometre (ERA, 2013).

- Total number of deaths and serious injuries per accident category (number)
- Relative number of deaths and serious injuries per accident category (number/train-kilometre)
- Breakdown of accidents according to different users/stakeholders of the railway system

Indicators for the financial impact assessment of accidents

- Total cost (in €)
- Unit costs (€/train-kilometre) for the number of fatalities and serious injuries, the cost of environmental impact, the cost of damage to rolling stock or infrastructure and the cost of delays resulting from accidents

The downgrading of a risk level (e.g., from 'non-desirable' to 'permissible') requires additional safety measures, which inevitably increase the investment cost, as well as the operational and maintenance cost of the railway system.

The measures taken by the railway authority, aiming at reducing the probability of an incident occurrence, are called *preventive measures*.

The measures that must pre-exist in order to reduce the impact of an incident and to make rational actions following the incident (e.g., in case of a train which is immobilised on the track due to breakdown, evacuation of the train and process of removal of passengers from the area of the incident) are classified as *management measures* (consequence containing measures, escape and rescue measures).

18.2 SIGNIFICANCE OF SAFETY IN RAILWAY SYSTEMS AND DIFFERENCES IN ROAD SAFETY

18.2.1 Significance of safety in railway systems

Each railway company tries to optimise the processes and output of its production, maintaining at the same time a high safety level for all the activities involved.

More specifically, a transportation system can benefit from the safe production of transport services in the following areas:

- In the level of service offered to the users
- In the economic efficiency of the system
- In the environment surrounding the system

Furthermore, in what concerns the railway system, safety is one of the major inherent advantages over its main competitor (i.e. private cars), according to the official international accident records. In this context, a railway company needs to focus on safety in order to obtain a significant share in the transportation market.

Railway safety constitutes a first priority target for the EU within the framework of the efforts targeted on the revitalisation of the railway sector. During the last 15 years, the EU published a series of directives specified on safety issues (the most representative is the Directive 2004/49) (EC, 2004a). Besides, EU established the ERA which is, among others, committed to initiating methods for the evaluation and the monitoring of the safety of railway systems in Europe in a unified and integrated manner (EC, 2004b).

18.2.2 Distinctions between railway and road safety

As a transportation system, the railway differs from the road vis-à-vis its three main constituents, namely railway infrastructure, rolling stock and railway operation. Consequently, there are distinct differences in terms of safety between the road vehicle and the train, both in terms of the characteristics of the incidents (type, severity) as well as in terms of the safety measures taken for their prevention and management.

Indicatively

- The railway is constrained to one degree of freedom. Owing to the impossibility of a train performing manoeuvres while moving, braking is the only option when faced with the risk of two trains colliding or a train colliding with an obstacle. However, because of the relatively low adhesion between the wheel and rail (steel/steel contact) and the greater braking load, the braking distance of a train is much greater than that of a road vehicle (see Tables 1.4 and 1.5). Therefore braking rarely prevents a collision. Hence it is of great importance for the railway to 'prevent' such accidents by taking the necessary measures in order to avoid a situation of collision.
- The railway's operational and constructional features increase the impact of the aerodynamic effects that are developed during train movement (high speed, long length, and large frontal cross section). These phenomena may have negative consequences on

the rolling stock, the passengers and other users of the system (e.g., passengers waiting or moving on the platforms) as well as on the staff working by the track. At the same time, due to the train's large lateral surface, it bears greater transversal wind loading, thereby making it more susceptible to overturning owing to crosswinds.
- The conventional road transport means cannot use the railway track because of its structure (rails/sleepers/ballast). Moreover, very often the landscape, in which the layout of the track is integrated, is inaccessible to road transport. Consequently, if a train is immobilised on the track, either due to a fault or due to an accident, the evacuation of passengers from the site of the incident and the provision of first aid is in many cases a particularly challenging operation.
- The rolling surface of roads, contrary to that of the railway track, is almost impermeable to water; therefore driving in icy conditions or during heavy rainfall is hazardous (risk of sliding or aquaplaning).
- A collision between two trains, or a collision between a train and an obstacle, is inevitably violent due to the train's large inertia. To mitigate this, engineers choose high safety factors during the dimensioning of the rolling stock. The vehicle's frame is designed with higher resistance and front vehicles are equipped with reinforced bumpers. Furthermore, several auxiliary mechanisms and automated systems ensure the smooth rolling and operation of the train in case of failure of certain functions, and inform the driver of potential problems.

18.3 CLASSIFICATION OF RAILWAY INCIDENTS

In general, the railway incidents can be classified into 10 main categories, as illustrated in the second column of Table 18.2 (Pyrgidis and Kotoulas, 2006).

Within each main category there are subcategories. The classification adopted here takes well into account the classification suggested by the European Directive for Safety.

According to the line segments on which they may occur, rail incidents are classified into the following categories:

- Incidents at civil engineering structures such as tunnels/bridges
- Incidents at stations/stops
- Incidents on the 'open track'
- Incidents at railway level crossings (RLCs)

18.4 CAUSES OF RAILWAY INCIDENTS

When it comes to railway incidents, special attention must be paid to the clear determination of the causes behind an event. The infrastructure manager or the railway company is expected to examine what triggered a particular incident in order to take corrective measures and subsequently mitigate the risk.

To that aim, the causes of railway incidents are grouped into three different levels, with regard to the 'source' event that triggered the chain.

Table 18.3 presents the various causes, according to the three aforementioned levels (Pyrgidis and Kotoulas, 2006).

A first-level cause might be one of the three main constituents of the railway system and/or a combination of them, as is the usual case.

First-level causes can also originate from external factors; these cases are classified in Table 18.2 under the group label *other miscellaneous incidents*. Typical examples include

Table 18.2 Main categories and subcategories of railway incidents

Main incident category	*Incident subcategory*
1 Vehicle derailments	• Derailment on the main track • Derailment on sidings
2 Train collisions	• Collision between trains (head-on, rear-on, head to flank, and broadside) • Collision between train and obstacle on the track
3 Separation of a train formation (splitting of coupled vehicles)	• Splitting of a passenger/freight train formation
4 Drifts by the rolling stock	• Pedestrian/animal drifts on track
5 Fires/explosions	• Fire on the train/in the tunnel/next to the track/in facilities • Explosion on the train/in the tunnel/next to the track/ in facilities
6 Incidents at RLCs	• Collision between train and road vehicle • Collision between train and truck • Collision between train and motorcycle • Pedestrian drifts at RLC area
7 Work accidents	• Incidents during working hours • Incidents during the return
8 Incidents involving hazardous goods	• Derailment/collision/splitting of a train formation carrying hazardous goods • Explosion in a wagon carrying hazardous cargo on route/during the process of maintenance works/during the cleaning process • Leak of toxic liquids/toxic vapors/radioactive fumes from the wagons
9 Other miscellaneous incidents	• Vandalism • Sabotage • Terrorist actions • Occupation of the track (by strikes) • Suicides and suicide attempts • Occupation of the track by waters/flooding of the facilities • Passenger(s) fall(s) from the platforms
10 Events able to cause one of the aforementioned incidents 1–9	• Violation of/incorrect application of the regulations by the staff • Technical failures in the railway infrastructure and/or in the rolling stock

Source: Adapted from Pyrgidis, C. and Kotoulas, L. 2006, An integrated system for the recording and monitoring of railway incidents, *6th World Congress in Railway Research (WCRR)*, 4–8 June, Montréal, Canada, Congress Proceedings.

natural phenomena such as large-scale earthquakes, heavy and continuous rainfall or snowfall and generally extreme weather conditions. Furthermore, terrorist attacks, vandalism, sabotage, suicide attempts and suicides, level crossing accidents (if the accident is caused by the road vehicle), etc. are also included.

Thus the first-level causes might be

- The railway infrastructure (track, civil engineering structures, track systems, track facilities and premises)
- The rolling stock (locomotives and trailer vehicles)
- The railway operation
- Combination of the above
- Causes external to the railway system

Table 18.3 Classification of causes of railway incidents by origin, in first, second and third level

Cause of incident – first level		*Cause of incident – second level*
1	Railway infrastructure	• Failure in the track superstructure • Failure in the track bed • Failure in tunnels/bridges • Failure in the earthworks/retaining walls • Failure in the signalling/electrification/ telecommunications systems • Failure in the level crossings • Failure in the track fencing
2	Rolling stock	• Failure in the car body/bogies • Failure in the axles/wheels • Failure in the doors/windows • Failure in the braking system • Failure in the electrical/electronic equipment • Failure in the coupling/buffering system • Failure in the motors
3	Operation	• Violation of the regulations • Misapplication of the regulations (e.g., red light violation, excessive speed, wrong switch setting, erroneous radio communication, wrong vehicle loading, and sudden braking/accelerating of the train) • Unprofessional condition of the employee (tired, drunk, etc.)
4	External causes	• Railway employee's sickness • Car driver negligence at a level crossing • Car driver inability due to alcohol or drug use • Car driver loss of control due to weather conditions • Terrorist attack • Vandalism • Sabotage • Suicide attempts • Suicides • Natural phenomena (earthquake, storm flooding, strong winds, snowfall, ice, and extreme weather conditions)
		Cause of incident – third level
	Failures	
	In the track superstructure	• Rail cracks • Rail defects • Track geometry defects • Sleeper cracks • Error in the track alignment geometry (insufficient cant, insufficient length of transition curves, etc.) • Soft ballast • Insufficient ballast • Ice • Slackened or inadequate fastenings • Bad condition of track switches and crossings
	In the track bed	• Ballast mouldering • Collapse of substructure

(*Continued*)

Table 18.3 (Continued) Classification of causes of railway incidents by origin, in first, second and third level

	Cause of incident – third level
In tunnel	• Lateral displacement • Water flooding • Narrow (inadequate) structure gauge • Insufficient aerodynamic behaviour • Structural failure
On bridges	• Structural failure • Pillar displacements • Aggrading • Rusting of metallic elements • Undermining of the pillars
In earthworks and retaining walls	• Embankment settlement • Falling rocks
In the signalling system	• Burnt lamps • Cable failures • Relay failures/malfunctions • Power supply failure • Track circuit failures • Faults in tele-commanding • Defects related to rail cracks • False signal indication • Miscellaneous external causes
In the electrification system	• Insulator failures • Defective station equipment • Short circuits • Power failure • Other external causes
At level crossings	• Broken barriers • Defect in the train announcement system • Insufficient/lack of road signs • Insufficient lighting
In the car body	• Fracture/distortion of the vehicle frame • Fractured or damaged container • Fractures/cracks in wagons with hazardous content
In the bogies	• Fracture of bogie frame • Distortion of suspension springs • Wrong design of suspension system
In the wheelset	• Fracture of axles • Axle-box overheating • Wheel cracks (flange, tread) • Wheel wearing (flange, tread)
At the doors	Door malfunction
At the windows	Broken windows
In the coupling system	• Damaged/broken couplers • Poor coupling
In the motors	Motor failure
In the vehicle's electrical/ electronic equipment	• Cab-signal failure/defects • GSM-R failure/errors in the radio communication
In the braking system	Braking system malfunction/failure

18.5 SAFETY IN CIVIL ENGINEERING STRUCTURES

18.5.1 Railway civil engineering structures and related incidents

The term 'civil engineering structures' in rail transportation systems describe all the structures built along the track, with the purpose of integrating the track layout in areas with difficult topography and/or sensitive environment. They must ensure the safe rolling of trains while being harmoniously integrated into the existing environment.

Accidents taking place on civil engineering structures are usually the gravest and have the worst consequences, including several fatalities, due to the profound difficulty for all escaping or rescuing operations (on bridges, in tunnels, etc.) as well as the high cost of the applied mitigation measures.

Virtually, all incidents taking place in the 'open track' can also occur on civil engineering structures. Table 18.4 shows the incidents involved which require special handling from the operator of the railway system (Pyrgidis et al., 2005; Pyrgidis and Kehagia, 2012).

18.5.2 Safety at railway bridges

The main cause of accidents on bridges is crosswind (Fujii et al., 1999). A characteristic example is the accident on Amarube Bridge in Japan, in 1986, where seven railway vehicles were lifted by the wind and pushed over the bridge, causing six deaths.

It has been proven that when the speed of the wind exceeds 25 m/s, the transversal and vertical acceleration of the bridge deck increase alarmingly, rendering its crossing by a train extremely unsafe (Li et al., 2005; Xia et al., 2006, 2008).

The thicker the bridge deck, the greater the transversal force coefficient.

In the event of an untoward incident (collision of trains, derailment, and immobilisation of train on a bridge), the presence of a civil engineering structure above ground level or

Table 18.4 Incidents on rail civil engineering structures requiring special management

Civil engineering structures	*Railway track*
Bridges	• Train derailment and falling from the bridge (for various reasons) • Pedestrians dragged along by rolling stock • Train derailment due to strong crosswinds and falling from the bridge • Workers dragged along by rolling stock (due to aerodynamic phenomena) • Train immobilised on a bridge
Tunnels	• Fires or emission of toxic gases and smoke inside the tunnels • Work accidents • Passengers' discomfort with regards to noise • Shattering of window pane on train carriage • Train immobilised inside a tunnel
Road overpasses	• Loss of road vehicle control and falling from the road deck on the track • Object falls from the road on the track
Embankments	• Train derailment due to strong crosswinds and overtopping from the top of the embankment • Violation of permitted limit of track defects due to embankment settlement – train derailment • Train immobilised on a high embankment
Cuttings	• Collision between train and obstacle on the track • Accident due to landslide • Rock fall • Train immobilised in deep cutting

Figure 18.1 Anti-derailment protective check rails. (Photo: A. Klonos.)

water, as well as the narrow and constrained space, further complicate the evacuation of the passengers and the access of rescue services and increase the severity of the incident.

In all cases, the placement of anti-derailment protective checkrails along the track should be implemented (Figures 18.1 and 18.2).

Table 18.5 illustrates the safety measures used on railway bridges.

Figure 18.2 Anti-derailment protective check rails. (Photo: A. Klonos.)

Table 18.5 Mitigation measures implemented on railway bridges

Category of measures	*Railway bridges*
Preventive measures	• Wind barriers and drapes • Anemometers • Footways for workers
Consequence containing measures	• Anti-derailment protective checkrails
Escape and rescue measures	• Footways for evacuation of passengers in case of an emergency • Construction of emergency exits for evacuating to a safe place • Safety manholes

18.5.3 Safety in railway tunnels

The most severe accidents that can take place inside tunnels involve fire and, consequently, emission of smoke and toxic gasses. Additional problems are generated by the presence of aerodynamic pressures or reduced ventilation (in the case of diesel locomotives). When entering tunnels, high-speed trains suffer from a sudden shift in the pressure conditions around them, and passengers experience a sudden reduction in their acoustic comfort. Moreover, cracking of windowpanes may take place due to this pressure. In such cases (train collisions, fire, immobilisation of train inside tunnel, etc.), the presence of a civil engineering structure below ground increases the severity of possible consequences. Especially in long tunnels – where rescue services are faced with the urgency to access the incident site as quickly as possible and initiate fast evacuation – the situation can get really difficult.

Two construction types – regarding railway tunnels – have prevailed internationally: the single-bore double-track tunnel (one tunnel with two tracks) and the twin-bore tunnel (two tunnels with single track).

The main advantage of a twin-bore track tunnel is that there is no risk of head-on collision and there are no aerodynamic problems occurring from trains moving in opposite directions; moreover, there is a high degree of protection in case of a fire event. On the contrary, the single-bore double-track tunnel has the major advantage of lower construction cost and clearly reduced aerodynamic effects (Maeda, 1996).

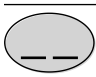
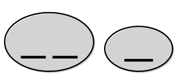
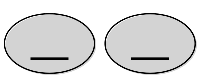
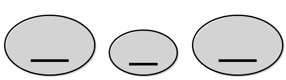
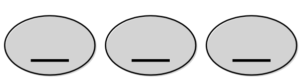
Table 18.6 presents the relative risk values for various tunnel configurations compared with the risk value of a single-bore double-track tunnel. Evidently, the three single-track tunnels and the twin bore with an auxiliary tunnel present lower risk but bare significantly higher construction cost.

Table 18.7 presents the safety measures taken in railway tunnels.

18.5.4 Safety at road overpasses

Accidents at road overpasses and railway underpasses usually occur as a result of objects falling from the road bridge. The most severe incidents occur when the pillars supporting the road overpass are not adequately reinforced and protected against the impingement of a derailed train.

Table 18.6 Relative risk value for various tunnel configurations compared with the risk of a single-bore double-track tunnel without auxiliary tunnel

	Tunnel configurations	*Relative risk value*
	Single-bore double-track tunnel	100
	Single-bore double-track tunnel + auxiliary tunnel	80
	Twin-bore tunnel	50–60
	Twin-bore tunnel + auxiliary tunnel	40
	Three single-track tunnels	<40

Source: Adapted from Diamantidis, D., Zuccarelli, F. and Westha, A. 2000, Safety of long railway tunnels, *Reliability Engineering and System Safety*, 67c, 135–145.

Table 18.7 Mitigation measures implemented in railway tunnels

Category of measures	*Rail tunnels*
Preventive measures	• Control for surveillance of tunnels • Hot-box detection devices placed at tunnel entrance (Figure 18.3) • Avoidance of switches and crossings inside tunnels
Consequence containing measures	• Fire-resistant materials and structures • Automatic fire, smoke and toxic gas detection systems • Installation of fire extinguishing system • Ventilation system to control heat and smoke • Water supply • Emergency power supply • Measures to reduce aerodynamic effects
Escape and rescue measures	• Escape routes and emergency exits • Safety and evacuation lighting • Emergency exits leading to ground (Figure 18.4) • Emergency contacts • Auxiliary tunnel for single-bore double-track tunnel • Cross-connections between tunnel tubes • Staff refugees

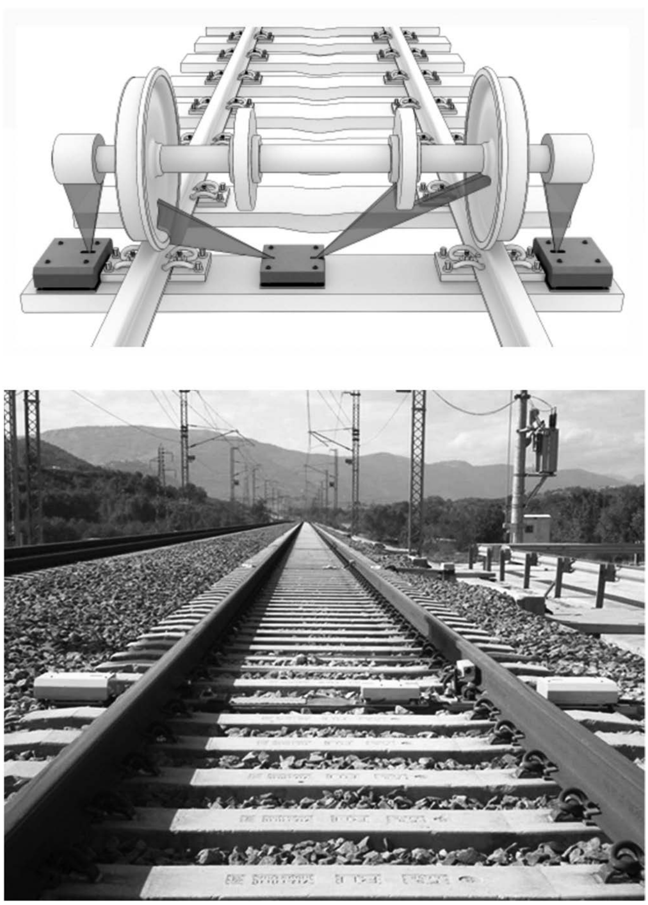


Figure 18.3 Hot-box detection devices. (Online image available at: http://www.mermecgroup.com/inspection-technology/hot-box-detector-/418/1/hot-box-detector-.php (accessed 12 June 2015).)

Figure 18.4 Escape exit, Kallidromos Tunnel, Greece.

Table 18.8 Mitigation measures implemented at road overpasses

Category of measures	*Road overpasses (rail underpasses)*
Preventive measures	• Proper design of alignment geometry and cross section of the road • Pedestrian facilities on the road bridge • Signing and road markings on the road
Consequence containing measures	• Safety vertical parapets and barriers • Protective wall for road bridge supporting columns • Horizontal protective grid along the bridge (outside the bridge deck) (Figure 18.5)

Table 18.8 presents the safety measures taken at road overpasses constructed along a railway line.

18.5.5 Safety on embankments

In the construction of embankments for civil engineering structures, particular attention must be paid to their height (lower height relative to road embankments) (Pyrgidis et al., 2005) and to their compactness and coherence so as to avoid settlement effects (Selig and Waters, 1994).

Figure 18.5 Vertical and horizontal protection along the road bridge (outside the bridge deck), Vienna, Austria. (Photo: A. Klonos.)

Table 18.9 Mitigation measures implemented on rail embankments

Category of measures	*Rail embankments*
Preventive measures	• Drainage, protection from groundwater and rain water • Reinforcement of earth embankment's foundation • Monitoring of movements/displacements by optic fibres • Wind barriers and anemometers
Escape and rescue measures	• Splitting the railway track into 'safety zones'. Connecting 'safety zones' with the road network. Contingency plans for evacuating to safe areas

The geometric track defects (cyclic top, twist, etc.) that may occur along the track are particularly dangerous for the movement of trains and in any case they are dangerous for the operation of the railway system.

Table 18.9 presents the safety measures taken on rail embankments.

18.5.6 Safety in cuttings

The most common accidents on the railway cuttings are caused by rocks which fall from the slopes of cuts on the track. In this case there is a high risk of collision of the passing train with the obstacle on the track, given that the presence of the obstacle may not be perceptible promptly and that the conflict cannot be avoided by emergency braking of the train.

Table 18.10 presents the safety measures used on rail cuttings.

18.6 SAFETY AT RAILWAY STATIONS

When a train runs through a railway station at a certain speed, the generated airflow can disturb people standing on the platforms (passengers, station personnel, etc.). Reducing the speed of the train when moving next to platforms, or keeping a minimum distance from the track when standing on the platforms, can mitigate the problem.

For instance, the British Railways apply a minimum passenger safety distance of 1.50 m from the edge of the platform when the trains' passage speed V_p exceeds 200 km/h. The respective distance for the station personnel is at least 2.00 m.

Table 18.11 presents the maximum permitted passage speed for a train, in case of human presence at a distance of 1 and 2 m, respectively.

A lot of platform-related accidents involve baby carriages, strollers and other wheeled equipment, susceptible to moving after the wind flow caused by a passing train. There are specific measures that counter these phenomena; the most common is installing ventilation systems above the platforms in order to weaken the generated wind flow and reduce its force and speed.

Table 18.10 Mitigation measures implemented on rail cuttings

Measure type	*Rail cuttings*
Preventive measures	• Protection from rock fall (fences and catch nets, protective gullies, rock-trap ditches, and retention walls) • Protection against slope slip • Track guard presence (foot patrols)
Escape and rescue measures	• Splitting the railway track into 'safety zones'. Connecting 'safety zones' with the road network. Contingency plans for evacuating to safe areas

Table 18.11 Maximum permitted passage speed for trains in case of human presence at distances of 1 and 2 m, respectively

Wind force criterion	*Maximum permitted train passage speed for distance of 1 m (km/h)*	*Maximum permitted train passage speed for distance of 2 m (km/h)*
Beaufort scale level 5 (for passengers)	80–118	98–146
Beaufort scale level 7 (for personnel)	127–188	156–232

18.7 SAFETY ON THE 'OPEN' TRACK

18.7.1 Potential risks

All the incidents recorded in Table 18.2 can also take place in the 'open track'. Table 18.12 presents the potential risks entailed within train operation in the 'open track' as well as the applied mitigation measures.

The term 'potential risks' includes all the incidents that can occur during the operation, as well as the contingency actions that follow such incidents.

Table 18.12 does not include the following:

- All incidents that can be avoided if the technical specifications of the project are completely fulfilled (e.g., derailments due to insufficient geometrical design with regard to the track design speed)
- All incidents that can be avoided if track maintenance is properly carried out (e.g., derailments due to geometric track defects)
- All incidents that are caused by possible malfunctions in the electromechanical installations and the related equipment
- Incidents at RLCs (they are examined in Section 18.8)

18.7.2 Safety measures

The entire track should be split into subsections called 'safety zones'. This approach offers several possibilities

- Immediate detection of the incident's corresponding track section
- Quickly determining the nearest available emergency exits in all cases
- Operational and contingency planning and structuring for the railway system operators

Some of the suggested safety measures of Table 18.12 are analysed hereunder:

- *Emergency exits*
  The locations of emergency exits are chosen so as to optimally satisfy the following criteria:
  - They must lead to a safe place, away from the original source of risk, as well as from other ensuing consequent risks. Therefore, emergency exits and routes are usually placed next to the road network.

Table 18.12 Potential risks in the 'open track' and respective safety measures

Incidents	*Safety measures*
• Pedestrian/animal drifts on track	Fencing along the track, overpasses
• Collision between train and obstacle on the track • Objects fall from the road bridge above the track	Bridge safety barrier as well as horizontal protective grid along the bridge (outside the bridge deck)
• Rocks falling from cut slopes	Reduced speed, surveillance of the track (foot patrols)
• Rear-end collision between two successive trains (e.g., as a result of different running speeds)	Overpassing tracks, sidings
• Violent push, passengers/personnel fall from the platforms, mainly due to the aerodynamic effects of a run-by train	Train speed reduction, maintaining adequate safety distance from the lateral edge of platform, and installing ventilation systems at the platforms
• Train is derailed due to axle-box overheating	Hot axle-box detection
• Train is derailed and subsequently enters the road network	Metal beam crash barriers, reinforced concrete walls, etc.
• Train is derailed (overturned) following strong crosswind	Wind barriers and speed reduction
Actions	
• Passengers must evacuate the train and the incident site	Adequate space along the track at each side providing safe and efficient flow of passengers and personnel In case the track lies on high embankments or next to a cliff edge, lateral protection of the track can be ensured by railings, barriers, etc. Emergency routes and exits should also be clearly communicated to passengers through proper signs
• Passengers need to relocate to areas where first aid can be provided	Emergency exits leading from the track occupation zone to locations that are connected with the road network and which allow for medical and first-aid agencies to operate. Easy-to-access emergency exits. (In case of embankments, appropriately designed stairways are required to facilitate the passengers' descend from the track level. In case of earth cuts, appropriately designed stairways are required to allow passengers to ascend from the track to the fencing level.)
• Ambulances should be able to get as close as possible to the track	Emergency exits leading from the track occupation zone to locations connected with the road network
• Emergency personnel should be able to quickly and easily access the point on the track where the train was halted	Special track – road vehicles
• Planning for crossovers in the track layout in order to account for maintenance activities as well as optimisation of the operation in anticipated emergency cases	Crossovers

- They must ensure that these locations can be accessed by multiple directions. Thus – if present – locations near road junctions, upper and lower crossings of intersecting roads, etc. are usually preferred.
- They need to facilitate the deployment of rescue teams for passengers and personnel, as well as medical and technical services to remove the halted rolling stock, provide first aid and support in general.
- They must ensure minimum interaction with other civil engineering structures (such as bridges, etc.), especially in case they are still under construction; that would mean possible revision of the original design, delay in the projects and cost overruns.

The distance between consecutive emergency exits is determined by the system operator.

- *Markings:* Emergency exits should be properly marked along the line by signs communicating the direction to the nearest exit as well as the relevant distance.
- *Road and rail corridors separation measures:* Sometimes the distance between road and rail is particularly small, and this creates the need for additional protective measures in order to safeguard the system. These measures include
  - Protective security barrier (0.75 m high)
  - Reinforced concrete walls (1.15 m high)

The selection and configuration of the required separating elements depends on the altitudinal difference as well as on the horizontal distance between the transport corridors. Horizontal distance is defined as the distance between the outer rail and the edge of the security barrier or the auxiliary lane (or the traffic lane of the secondary road). Altitudinal difference represents the height difference between the rolling surface of the rail and the road running surface level.

18.8 SAFETY AT RLCs

Table 18.13 presents the causes of RLC incidents, grouped into three levels according to the 'source' event that triggered the chain.

In a railway level crossing collision incident, regardless of who provokes it – namely the road vehicle or the train – it is the vehicle's driver whose life is being threatened; what is more, only she/he can actually do something to prevent the accident from happening. Therefore, she/he is expected not only to adhere to the traffic regulations but also to double check carefully the crossing, even if the signals show 'go'.

RLCs constitute a location where rail and road traffic flows intersect. Given that their density in conventional speed railway networks around the world remains high, the RLCs are a neuralgic point of the railway system, both in terms of traffic safety and in terms of operation.

For these reasons, level crossings should be examined separately, since they constitute a major cause of railway accidents, and all the railway companies should employ a system to manage the safety level provided to their customers.

The term 'RLC management' represents the whole process /methodology that the railway infrastructure manager has to follow during the design, construction, operation and maintenance of RLCs, aiming at

Table 18.13 Classification of causes of railway incidents at RLCs, into first, second and third level of source

Causes of incidents at the first level	*Causes of incidents at the second level*
Railway infrastructure	• Failure of RLC equipment • Poor geometry/construction of the RLC
Railway operation	• Liability of the train driver • Liability of the level crossing guard
Causes external to the railway system	• Liability of the driver of the road vehicle • Liability of the pedestrian • Breakdown of the road • Act of terrorism • Vandalism • Sabotage • Extreme weather conditions
Causes of incidents at the third level	
Failure of RLC equipment	• Broken barriers • Failure of the train announcing system • Burnt signal lamp • Power cut-off • Mechanical breakdown of barriers, sound and signal light activation
Faulty geometry/construction	• Reduced visibility due to obstacles • Insufficient lighting – improper lighting • Problematic geometry of intersecting road • Lack of road or railway signposting • Poor state of signposting • Poor state of road surface
Liability of the train driver	• Wrong maneuver • Violation of signalling rules
Liability of the level crossing guard	• Wrong maneuver • Negligence, recklessness
Liability of the driver of the road vehicle	• Violation of the traffic code • Poor judgment • Incapacity due to the influence of alcohol or drugs • Tiredness, distraction • Halted vehicle on the track
Liability of the pedestrian	• Distraction, not paying attention • Indisposition • Bad judgment • Incapacity to react due to the influence of alcohol or drugs • Fatigue

Source: Adapted from Pyrgidis, C. and Sarafidou, M. 2007, Railway level crossing management system, in *9th International Congress Railway Engineering* 2007, 20–21 June 2007, London, University of Westminster, Congress Proceedings.

- Limiting the chance of an accident at RLCs to the minimum extent
- Securing the passage of trains through the RLC location at the desired speed
- Maintaining the cost of operation of the RLC and of the maintenance of its equipment at acceptable levels

Figure 18.6 illustrates an integrated system for the management of the RLCs. This system includes the following managing tools (Pyrgidis and Sarafidou, 2007):

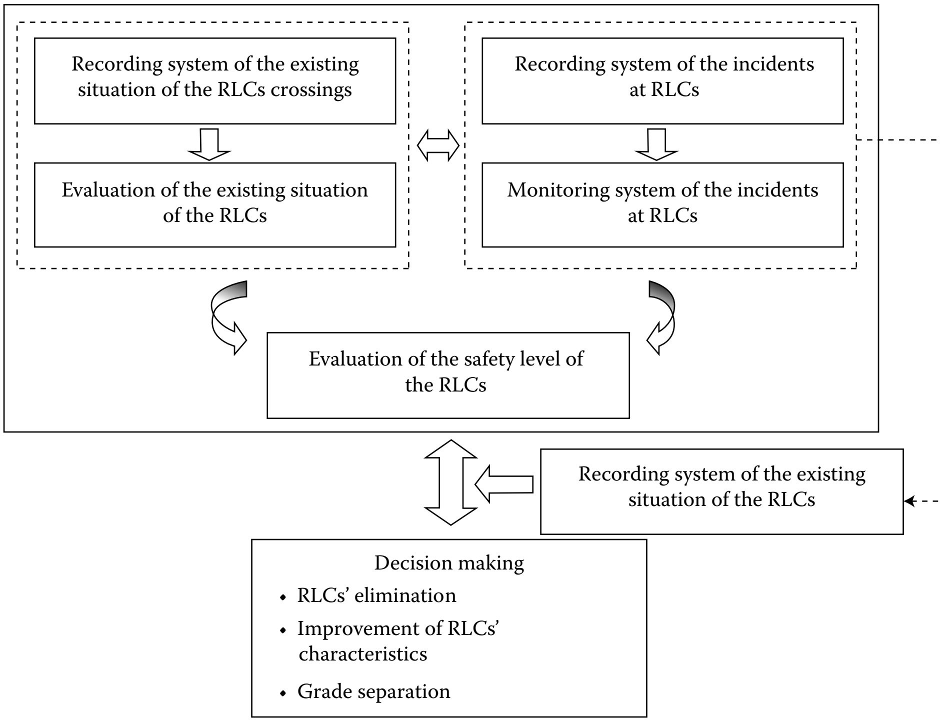


Figure 18.6 Integrated RLCs management system. (Adapted from Pyrgidis, C. and Sarafidou, M. 2007, Railway level crossing management system, in *9th International Congress Railway Engineering 2007*, 20–21 June 2007, London, University of Westminster, Congress Proceedings.)

- Recording system of the existing situation of the network's RLCs (i.e. of their functional and structural characteristics)
- System for evaluation of the existing situation of RLCs (equipment, structural and functional characteristics)
- Recording system of the incidents at RLCs
- Monitoring system (through indicators) of the incidents
- Evaluation system of the safety level of the RLCs of the network

The interaction of these five 'management tools' can help the infrastructure manager to take decisions regarding the interventions to be made on the network's RLCs, in order to minimise the occurrence of accidents and to help the operation of the network.

Those interventions are essentially three (Mallet, 1987; George, 1999):

- Elimination of RLCs
- Improvement of RLC characteristics
- Grade separation of the level crossings that are characterised as dangerous

The removal of an RLC and the shifting of road traffic flow to the next possible level crossing through the adjacent road, means essentially shifting the risk to the next crossing, and results in increased travel time for the road users.

Improving the construction and operational characteristics of an RLC includes

- Conversion of the RLC from passive to active*
- Upgrading the existing equipment of an active RLC
- Systematic maintenance of the equipment of the RLC
- Removal of the elements (e.g., advertising boards) that constitute potential distraction for the road vehicles' drivers and trains' drivers
- Installation of closed-circuit television (CCTV)
- Installation of automatic obstacle detection system
- Improvement of the visibility conditions
- Improvement of the lighting
- Improvement of the pavement

Finally, the conversion of an RLC to a non-level crossing is performed by constructing underpasses or flyovers (overpasses).

The cost of the equipment of an RLC is characterised by significant variations among countries, mainly due to the diversification of the technical specifications adopted in each country, and the differentiation of the individual characteristics of each network (e.g., single or double track and train speed). For example, the cost of automatic gates may be as low as €100,000 but it may also exceed €900,000.

Table 18.14 provides the cost values for various interventions at a passive RLC (Ioannidou and Pyrgidis, 2014).

When selecting the optimal solution, the following should/must be taken into account:

- Number of RLCs per line-kilometre
- Daily traffic moment†
- Statistical data related to accidents
- Possibility to rearrange the road network in the broader area of the RLC
- Land uses in the proximity of the crossing
- Cost of intervention

Table 18.14 Cost values for various alternative interventions at a passive RLC

Type of intervention	*Installation cost (in €)*	*Annual maintenance cost (in €)*
Installation of semi-automatic barriers at the RLC	370,000	2,300–5,700
Installation of automatic barriers at the RLC	570,000	2,300–5,700
Removal of the RLC	50,000–70,000 (+construction of road connection in parallel and near the track)	–
Conversion to overpass	3,200 per m^2	3,500

Source: Adapted from Ioannidou, A.M. and Pyrgidis, C. 2014, The safety level of railway infrastructure and its correlation with the cost of preventive and mitigation measures, *International Journal of Railway Research*, 1(1), 19–30, 2014.

* *Passive level crossing*: level crossing without any form of warning system or protection activated when it is unsafe for the user to traverse the crossing. *Active level crossing*: level crossing where the crossing users are protected from or warned of the approaching train by devices activated when it is unsafe for the user to traverse the crossing.

† *Daily traffic moment*: The number of trains moving on the track in both directions per 24 h multiplied by the number of passing road vehicles of all types in both directions of the crossing during the same 24-h period.

The final selection will result from a feasibility study (financial and socio-economic analysis). In many cases, eventual social pressure might be a determining factor.

At this point it should/must be emphasised that the application of such a system must always be accompanied with an organised effort from all bodies involved, with respect to information and to raising public awareness.

18.9 CORRELATION BETWEEN THE COST OF INTERVENTIONS AND THE SAFETY LEVEL IMPROVEMENT

18.9.1 General approach

One of the main issues that railway companies have traditionally dealt with is the amount of money they need to invest initially or during the system's operation in order to ensure a specific level of safety. Safety improvement is costly; however, what is not often known is its correlation with the required cost. The quantification of this correlation is difficult because it is defined by a number of factors, whose characteristics have not been specified yet.

For the correlation between the interventions' cost and the anticipated safety improvement two methods can be followed. These two methods are based on the two different definitions of railway safety which are given in Section 18.1 (Figure 18.7).

In the first method (which is based on the change of the value of the accident indicators), the aim is that the measures addressing incidents should assist toward the reduction of the selected accident's quantification indicator. In the second method (based on the change in the risk level), the aim is to assist toward the qualitative improvement of the initial risk level (Ioannidou and Pyrgidis, 2014).

The chart of Figure 18.8 illustrates the first six steps of the proposed methodology which are common to both methods.

As seen in the chart illustrated in Figure 18.8, regardless of the methodology that will be followed, the correlation between interventions' cost and anticipated safety improvement presupposes the following:

- Definition of the study area and, particularly, the 'level' of the railway system for which accidents are assessed (e.g., whole network, railway corridor, track section and railway system constituent).
- The approach per accident category and, for each accident category, per accident cause at first level at least.
- The costing of the accidents' consequences.

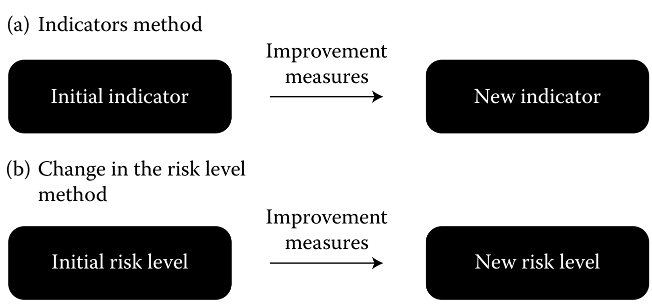


Figure 18.7 (a) and (b) Methods for the correlation between 'interventions' cost and anticipated safety level improvement.

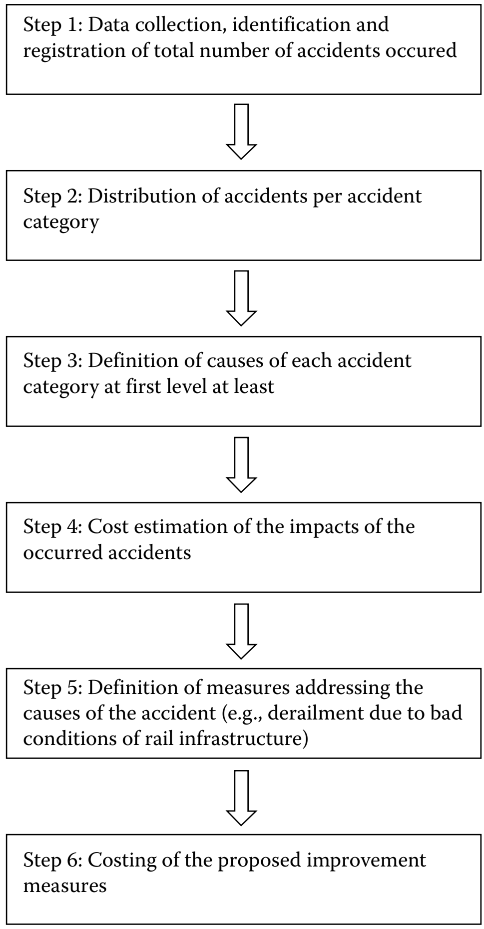


Figure 18.8 Proposed methodology for the correlation between interventions' cost and anticipated safety improvement – first common steps for the two methods.

- The definition of the type and extend of the measures to be taken. The combination of study area, accident category and accident cause will determine the relevant range of choices.
- The costing of the above measures.

Consequences of accidents include fatalities, injuries, material damage which covers both rolling stock and infrastructure damages, environmental damage and delays of service.

Concerning the cost of the consequences, it is stated that, according to the ERA, the value of preventing a fatality/casualty (VPF/VPC) amounts to €1,500,000, while the value of preventing an injury (VPI) amounts to €200,000 (2008 prices). According to a more up-to-date estimation (2013 prices) made by the British organisation RSSB (Rail Safety and Standards Board), the value of preventing a fatality (VPF) amounts to €2,230,000.

18.9.2 The change in the value of accident indicators

The proposed methodology uses an indicator which, depending on the incident, can be one of the indicators proposed by ERA, such as 'fatality risk indicator' or an indicator that involves the number of accidents for a specific accident category per vehicle-kilometre (e.g., number of derailments per train-kilometre and number of collisions per train-kilometre).

The correlation between the cost of interventions and safety improvement lies with the calculation of the amount of money that should be invested in order to reduce the current value of the indicator by a specific percentage or to set a new target value (i.e. the average rate applicable for EU countries for this incident category).

The first six steps that are common to both approaches (see Figure 18.8) are followed by the steps outlined hereunder:

Step 7: Assessment of the impact that the intervention's implementation has on the parameters that form the numerical expression and, as a result, the value of the indicator.
Step 8: New situation – calculation of new indicator's value.
Step 9: Correlation between the change in the indicator's value and the cost of interventions.

The assessment of the impact that the intervention's implementation has on the change of the indicator's value is the most difficult task. It can potentially be addressed by one of the three ways, namely:

- By appropriate prediction models
- By recording the number of incidents that have taken place or will take place at a particular constituent of the railway system for at least 5 years after the implementation of preventive and mitigation measures and comparing them with the previous situation
- Based on statistics from other networks with similar functionality

Concerning the prediction models, the railway infrastructure managers have at their disposal, according to the international literature, prediction models for railway incidents at RLCs, which they can use to assess the number of accidents that may occur at an RLC with the given functional and structural characteristics at a given time period (Coleman and Stewart, 1976).

For instance, the application of such a model to the Greek railway network led to the following findings (Morfoulaki et al., 1994):

- Installation of automatic road traffic signals and audible warnings in combination with automatic barriers reduces the number of accidents by up to 50% (upgrade of passive RLC to active).
- The increase of road traffic load by 50% leads to an increase in the number of accidents by 15%.
- The increase of rail traffic load by 50% leads to an increase of accidents by 12%.
- The increase of the number of railway tracks at the level crossing (from single track to double track) increases the number of accidents by 10%.
- The reduction of road traffic lanes leads to a slight reduction in the number of accidents.

Moreover, the installation of an obstacle detection system at an RLC with semi-automated gates results in the reduction of accidents by 30% (Woods et al., 2008), while the installation of lighting results in the reduction of accidents by 45% (Varma, 2009).

On the basis of experience from the installation of CCTV on the road network, it can be assumed that the installation of such a system at an RLC would result in a reduction of intentional infringements by 50% (Woods et al., 2008).

Table 18.15 provides the improvement that is brought about on the security level of the RLC specific measures (Saccomanno et al., 2006; Washington and Oh, 2006).

Table 18.15 Effectiveness of safety measures at RLC as a percentage of the reduction of accidents

Safety measures	*Effectiveness (%)*
'Speed humps' (measure for the reduction of speed on the road network)	36–40
Warning signage	0–50
Reduction of the road gradient at the location of the crossing	39–47
Increase of the distance from which the crossing is visible by the road network	0–50
Increase of the distance from which the crossing is visible by the railway network	10–41
Construction of a pedestrian crossing	0–50
Installation of lighting at the road network	15–45
Stop signs	35–46
Conversion of the RLC to non-level railway crossing	100
Road and railway network lighting	44
Removal of prohibition of the use of train audible warning system	53
Improvement of visibility	34
Improvement of the condition of the road pavement	48
Reduction of the speed limit for road vehicles	20

Source: Adapted from Washington, S. and Oh, J. 2006, Bayesian methodology incorporating expert judgment for ranking countermeasure effectiveness under uncertainty: Example applied at grade railroad crossings in Korea, *Accident Analysis and Prevention*, 38, 234–247.

Case study: Particular passive RLC

Incident type: accident

Accident category: accident at passive RLC

Special accident category: collision of a train with a road vehicle

Cause of accident: railway infrastructure – poor visibility

Used indicator: number of fatal accidents (each with at least one fatality) hence the number of fatalities in the long term of 25 years = 10 fatalities = 0.40 fatalities per year

Measure: installation of automatic barriers

Intervention cost: €570,000 (installation of automatic barriers) + €5,000
(annual maintenance cost)

Impact of measure implementation: reduction of fatal accidents and, therefore, of the number of fatalities by 50% (Ioannidou and Pyrgidis, 2014)

New indicator' value: Five fatalities over 25 years – 0.20 fatalities per year

Cost of fatalities: €836,000 × number of fatalities + €760,000/year (fixed premiums) (ERA, 2009)

Economic life period of barriers = 25 years

Results of cost–benefit analysis: cost–benefit factor = 4.761069 ≫ 1 (25-year assessment period, 5.5% discount rate) (Ioannidou and Pyrgidis, 2014)

18.9.3 The change in the risk level

In this method, the correlation between the cost of interventions and safety improvement lies with the assessment of the money that must be invested in order to change the current level of risk of a railway system to a lower one or to a desired level. This change can only be made by changing the frequency of accident occurrence, by altering the severity of accidents or, finally, by a simultaneous change of both.

The first six steps that are common to both approaches are followed by the steps outlined hereunder:

Step 7: Classification of the accidents' frequency per accident category and causing source. For this process, a specific methodology is proposed in Section 18.9.3.1.
Step 8: Classification of the accident's severity per accident category. For this process a specific methodology is proposed in Section 18.9.3.2.
Step 9: Definition of risk level for each accident in combination with the frequency and severity, as defined in steps 7 and 8.
Step 10: Assessment of the intervention's impact on the accident's frequency and severity. Classification of the new accident's frequency and severity.
Step 11: New situation – calculation of the new risk level.
Step 12: Correlation between the results of the cost–benefit analysis and the new risk level.

The approach that is based on the change in the risk level is related with more problems in comparison with the approach that is based on the change in the value of incident indicators. These problems relate to the following:

- The quantification of six categories as proposed in Table 18.1 regarding the frequency of incident occurrence. The key questions raised are the following:
  - What is the value of each frequency category, what are its measurement units and which time period does it refer to?
  - Is the value of the frequency that characterises each frequency category the same for all accident categories?
  - Is there a distinction depending on the cause of the accident?
  - Is there a distinction depending on the category of railway system (metro, tram, high-speed trains, suburban trains, etc.)?
- The quantification of four categories proposed in Table 18.1 regarding the severity of incidents. The key questions raised are the following:
  - How is each category of severity defined?
  - Do the various accident categories belong uniquely to a particular category of severity?
- The assessment of the impact that the application of specific measures have on the value of their frequency of occurrence. This assessment can be done for the three measures, already outlined and for the case of the indicators methodological approach.
- The assessment of the impact that the application of specific measures has on the value of severity of the incidents' impacts.

18.9.3.1 Characterisation of the frequency of a particular incident

The quantification of frequency is an issue that remains under investigation. Literature review (Ioannidou and Pyrgidis, 2014) attempts to approach this topic. More specifically, a methodology is proposed in order to set quantitative limits of frequency categories as listed in Table 18.1 for each accident category. The main indicator used in order to set the values for frequency categories is the average number of accidents per accident category which have occurred at a large number of representative networks (e.g., the EU countries).

Table 18.16 Indicative percentages that form the values for the various frequency categories

Frequency category	*Indicative percentage so as to set the values*
Frequent	40% increase of the average
Possible	20% increase of the average
Occasional	Average number of accidents per accident category
Unusual	20% decrease of the average
Rare	30% decrease of the average
Unlikely	40% decrease of the average

Source: Ioannidou, A.M. and Pyrgidis, C. 2014, The safety level of railway infrastructure and its correlation with the cost of preventive and mitigation measures, *International Journal of Railway Research*, 1(1), 19–30, 2014.

It is expressed in different measurement units depending on the accident category and has a specific year as a reference. The average is considered to be the value for the 'occasional' frequency category while as the values for the other categories are based on the average, by increasing or decreasing it. An indication of the recommended percentages is provided in Table 18.16.

On the basis of the above, the steps of the proposed methodology for a specific accident and for a given year are as follows:

1. Assignment of the accident to the appropriate category
2. Collection of the necessary data so as to allow for the calculation of the average number of accidents
3. Calculation of the average for the accident's category, by using the measurement unit that corresponds to the particular accident category
4. Setting the average as value for the 'occasional' frequency category
5. Calculation and setting of the values of other frequency categories using Table 18.16 for the specific accident category
6. Determination of the accident's frequency category on the basis of the position of its average value in Table 18.16

In many cases, the proposed methodology should be further specialised in order to address a particular cause of occurrence for each incident (e.g., derailment [incident] and infrastructure [cause]).

18.9.3.2 Characterisation of the severity of a particular incident

Ioannidou and Pyrgidis (2014) attempt to approach the topic of incident severity in two ways.

The first approach involves the incident category (as defined in Section 18.3, Table 18.2) with its usual consequences. More specifically, it is considered that some incident categories have, in most cases, catastrophic consequences, such as loss of human lives. The classification of severity proposed in this research based on the incident categories is presented in Table 18.17.

Derailments and collisions are two incident categories which, regardless of the cause of their occurrence, in many cases, cause fatal accidents and/or multiple severe injuries and/or severe environmental impact, and/or extensive material damage; hence, they are classified into the 'catastrophic' severity category. Incidents at RLCs and incidents caused

Table 18.17 Definition of severity based on incident category

Severity categories	*Accident categories*
Catastrophic	Derailment of trains Collision of trains
Severe	Accidents at level crossings Accidents to persons involving rolling stock in motion
Low severity	Fires in rolling stock
Negligible	Other accidents

Source: Adapted from Ioannidou, A.M. and Pyrgidis, C. 2014, The safety level of railway infrastructure and its correlation with the cost of preventive and mitigation measures, *International Journal of Railway Research*, 1(1), 19–30, 2014.

by rolling stock in motion usually cause those impacts recorded in the 'severe' incident category, and are classified under the 'severe' category. For the same reasons, fires are classified under the 'low severity' category although in some cases their consequences can be catastrophic.

The second approach regarding severity involves its quantification based of the actual consequences of incidents that have occurred. A key indicator in order to form the values for each severity category of each incident category is the average of their consequences (at national or at international level). Depending on the average of their consequences, incidents are classified under one of the four categories, namely catastrophic, severe, low severity and negligible as already defined in the above.

Case study: Accidents at RLCs at network level

It is assumed that the selected preventive measure results in a decrease in accidents occurrence by 50% (Morfoulaki et al., 1994).

Incident type: accident
Accident category: accident at RLC
Total number of accidents per year = 30
Number of fatal accidents per year = 8
Total number of RLCs = 1,275
Number of active crossings (60%) = 765
Number of passive crossings (40%) = 510
Assumption: 20% of the accidents occur in the active level crossings. Hence:
Number of accidents/fatal accidents per year in the active crossings = 6/1
Number of accidents/fatal accidents per year in the passive crossings = 24/7
Total length of track = 2,500 km
Accident frequency: used indicator; number of accidents at RLCs per RLC per track-km = 9.41E-6 accidents per year
Average number of indicator (EU): 6.68E-6
Classification of frequency: possible
Classification of severity: severe
Risk level: non-permissible

Assumption: all accidents concerning passive crossings (24) occurred in 70 passive crossings (out of 510)

Measure: installation of 70 automatic barriers

Intervention cost (per gate): €570,000 (Installation of automatic barriers) + €5,000 (Annual maintenance cost)

Total cost (70 gates) – Present Value: €44,853,095

Impact of measure implementation: Reduction of accidents by 50% *only* in the passive crossings (all kind of accidents)

NEW SITUATION

Number of accidents/fatal accidents per year in the active crossings = 6/1

Number of accidents/fatal accidents per year in the passive crossings = 12/3

New accident frequency: new indicator's value: 5.65E-06 accidents per year

New frequency category: occasional

New severity category: severe

New risk level: non-desirable

Benefit – Present Value: 47,323,281

The change of risk level from non-permissible to occasional presents a cost/benefit ratio of 1.055 based on the saving of 7 – 3 = 4 lives per year

Net Present Value (NPV): 2,470,187

Internal Return Ratio (IRR): 6%

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Chapter 19

Railway and the natural environment

19.1 NATURAL ENVIRONMENT OF THE RAILWAY

Rail transport systems can move at grade, underground and above the ground (elevated). In this context, the constituents of the natural environment which these systems' infrastructure and operation affect are the air, the ground surface and the subsoil.

Specifically, the air is affected by

- The pollution caused mostly by air pollutants, which are emitted during the operation of diesel trains (air pollution) and during the production of electric power (electric trains)
- The noise emitted during the trains' operation (acoustic annoyance)

The ground surface, as well as anything that exists or moves on it (humans, flora, fauna, constructions, etc.), in general, are affected by

- The noise and vibrations which are caused during the running of trains on the rails, and are transmitted through the ground to neighbouring constructions (ground-borne noise and vibrations)
- The impacts caused to other means of transport, due to the land take by the railway infrastructure
- The changes caused to the landscape, and especially for the urban means of transport, and to the space (integration into the topographic relief/surrounding built environment)
- The resulting change in the aesthetics of the surrounding area, resulting from the presence of the new means of transport (visual annoyance)
- The disturbance of various activities of local residents and particularly for the urban means of transport, the disruption of continuity in the urban space
- The disturbance of the ecosystems

The subsoil and the underground water are affected by

- The pollution, which is caused by waste and pollutants released by railway vehicles during their movement in the main traffic lines, or derived from various activities that take place in the facilities and premises, the waste of which penetrate the soil (soil and water pollution)
- The vibration that is transmitted to the soil through the track superstructure

Finally, all three constituents of the natural railway environment, and, in general, the whole planet are affected by the climate change caused by the so-called greenhouse gases emitted by the means of transport.

In the broader context of environmental sustainability in addition to the above-mentioned effects, energy consumption is of particular importance.

All the above essentially compose the interface between the railway and the natural environment, and are described in more detail in the following sections. The land take, the ground-borne noise and vibrations, the disruption of continuity of the urban space, the changes in land uses and land values, and the integration of the railway infrastructure into the urban environment can be described as the interface between the railway and the so-called built-up environment. The land take has been examined in Sections 1.5.1 and 1.5.3.

19.2 ENERGY CONSUMPTION

19.2.1 Definition: Units expressing energy consumption

Energy is defined as the capacity of an object or a system to produce work. Railway systems move by using mainly two sources of energy: oil, in the form of diesel fuel (diesel trains), and electricity (electric trains). Diesel locomotives convert thermal energy to kinetic energy, while electric locomotives convert electrical energy to kinetic energy.

The quantities commonly used for the expression of energy consumption of the railway are displayed in Table 19.1, and the relationships that link the expression units with each other in Table 19.2.

19.2.2 Energy-consuming railway activities

The energy-consuming activities of the railway system are the circulation of trains and the railway facilities/premises (stations, depots, etc.).

With regard to the operation of trains, energy is needed to meet the six basic operations, and specifically

- Acceleration of the train
- Traction of the train
- Train movement on track sections with longitudinal gradient

Table 19.1 Units expressing energy consumption for passenger and freight railway transport

Passenger transport	*Freight transport*
kW h/passenger-seat	–
kW h/passenger-kilometre (kW h/pkm)	kW h/tkm
MJ/passenger-seat	–
MJ/passenger-kilometre (MJ/pkm)	MJ/tkm
Petrol (L)/passenger-seat	Petrol (L)/tkm
BTU/passenger-kilometre (BTU/pkm)	BTU/tkm

Table 19.2 Relationship between units that are used to express energy consumption

1 MJ = 0.2778 kW h
1 BTU = 2.931×10^{-4} kW h
1 L petrol = 9.7 kW h

- Supply of the control systems
- Lighting, heating, cooling and ventilation of the vehicles
- Transmission of power through the electric network to the driving wheels, in the case of electrification

With regard to the operation of the stations, it should be highlighted that, in railway networks with a large number of stations, the energy consumed for the operation of lifts, escalators, as well as lighting and heating of the stations, should comprise up to 20% of the total energy consumed by the railway system (UIC, 2008).

Finally, energy is consumed during both the production and the distribution of the final energy that is being used for the operation of the various activities of the railway system.

19.2.3 Special features of each railway system category

Figure 19.1 displays the energy consumption per category of passenger train, and for different traction systems.

It should be noted that regional trains consume more energy than interurban trains, and that electric trains consume much less energy than diesel trains (approximately 1/3).

The chart of Figure 19.2 displays the relationship between energy consumption and speed for various train categories.

By evaluating the above data, it can be stated that conventional-speed trains, running at speeds V < 200 km/h, consume 0.050–0.130 kW h/pkm (passenger-kilometre).

According to Network Rail, 2009, energy consumption for European high-speed trains ranges between 0.034 and 0.041 kW h/pkm, while speed ranges between 180 and 350 km/h. The high increase in the speed of trains does not result in a corresponding increase in energy consumption. One of the reasons behind this is the fact that the average distance of intermediate stops is lower than that of the conventional-speed railway.

Concerning urban railway systems, the metro is an energy-consuming system, since the train movement, as well as the operation of the auxiliary train and station facilities equipment, is performed through the supply of electrical energy.

The tramway is characterised by comparatively lower energy consumption, with about 50% being lost during braking. The use of energy-storage systems saves energy. Lighter vehicles allow for lower energy consumption (UIC, 2008).

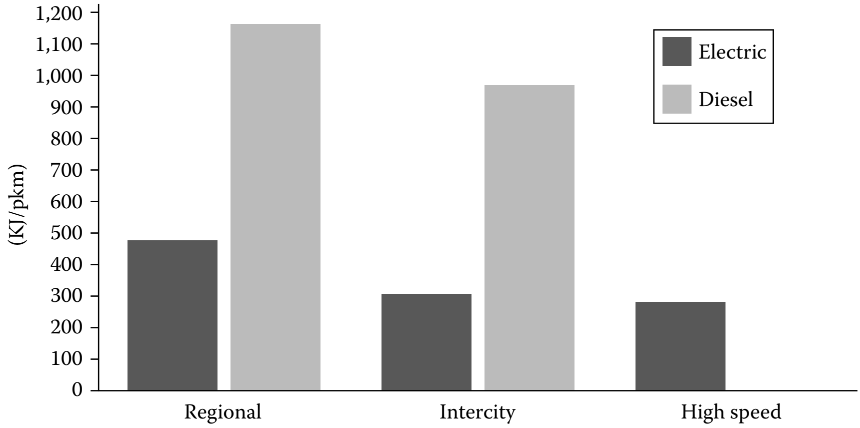

Figure 19.1 Energy consumption of passenger railway transport per train category and traction type, 2005. (Adapted from UIC. 2012, *Energy Consumption and CO_2 Emissions, Railway Handbook*, 2012.)

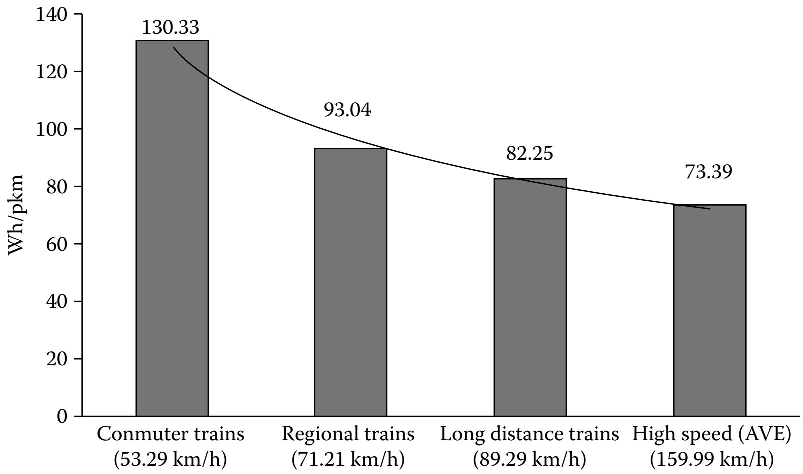

Figure 19.2 Energy consumption for various categories of railway systems per ascending order of average speed. (Adapted from Garcia, A. 2010a, *High Speed, Energy Consumption and Emissions*, UIC, 21 December 2010.)

Table 19.3 displays the range of fluctuation in the values of energy consumption for various categories of railway systems.

19.2.4 Measures for energy consumption reduction

The total energy consumption for European railways was reduced by 25% from 1990 to 2010, despite the fact that traffic increased. Specifically, in the field of passenger and freight railway transport, energy consumption was reduced by 13% and 18%, respectively, during the 20-year period of 1990–2010 (UIC, 2012).

The methods and techniques for reducing energy consumption consist of

- *Energy-efficient driving*: The training of engine drivers, combined with a consultation system inside the driver's cab during the train's movement, can reduce fuel consumption by 5%–20% (Veitch and Schwarz, 2011).
- *Aerodynamic design of trains and reduction of their weight*: In Japan, improvements in the design of the nose of the Shinkansen trains and reductions in vehicle weight have resulted in the reduction of energy consumption by 40%, although speed has increased (Veitch and Schwarz, 2011).

  The reduction of weight is of particular importance in urban railway systems (metro, tramway, and suburban railway) that make a lot of stops. The large weight of the trains, combined with frequent startups, leads to high energy consumption. Carbon

Table 19.3 Range of fluctuation in the values of energy consumption for various categories of railway systems

Interface with the environment	*High-speed railway*	*Conventional-speed railway*	*Metro*	*Tramway*
Energy consumption (kW h/pkm)	0.035–0.100	0.050–0.130	0.015–0.055	0.008

fibres are the new composite material that has been increasingly applied recently and contributes toward the construction of lighter vehicles (see Chapter 20).

- *Systems for the storage of the energy that is diffused during braking*: Energy storage can be performed by batteries, supercapacitors, flywheels and superconducting magnetic energy-storage systems.
- *LED eco lighting*: This is a very low-energy lighting solution, which consumes up to 3 times less energy than conventional systems.

19.3 AIR POLLUTION

19.3.1 Definition: Expression units of air pollution

Air pollution is defined as any condition where there are substances in the ambient air, in much higher concentration than the normal, which may cause measurable effects to humans and, in general, to organisms, vegetation or, further, to constructions/materials.

The impacts of the railway, as well as of transport systems in general, to air pollution are divided into two main categories:

- Local pollution, which includes all forms of air pollution that affect the quality of ambient air and health on a local scale
- General pollution (of the planet), which involves the destruction of the ozone layer, and the 'greenhouse effect'

The main pollutants emitted by various means of transport and producing local pollution are carbon monoxide (CO), nitrogen oxides (NO_X), hydrocarbons (HC), solid particles (TSP), sulphur dioxide (SO_2) and various mineral traces.

The main gases of the atmosphere that cause general pollution are carbon dioxide (CO_2), methane (CH_4), nitrogen oxide (N_2O), chlorofluorocarbons (CFCs), tropospheric ozone and vapours at high altitude.

The quantities commonly used in the expression of local air pollution caused by the railway are displayed in Table 19.4.

General air pollution is measured based on the generated carbon footprint. The carbon footprint is defined as *the total sets of greenhouse gas emissions caused by an organization, event, product or person* and is measured in emitted tons of carbon dioxide equivalent (t CO_2e) (Carbon Trust, 2012).

19.3.2 Railway activities causing air pollution

Air pollution is caused during the operation, as well as during the implementation of a railway system.

Table 19.4 Indices for the expression of local air pollution caused by railway systems

Passenger transport	*Freight transport*
kgr or gr CO_2/pkm or seat	kgr or gr CO_2/tkm
gr NO_x/pkm or seat	gr NO_x/tkm
gr NMHC/pkm or seat	gr NMHC/tkm
gr SO_2/pkm or seat	gr SO_2/tkm

During the operation of a railway system, there are air pollutants only in case of diesel traction, whereas in electrification, ambient air in the area close to the railway network is affected only by the use of polychlorinated biphenyl in the power converters of the electrical equipment of the rolling stock and the substations. In contrast, air pollutants are produced in the area of electrical power production.

During the construction of railway projects, the sources of air pollution are the following:

- Emissions of air pollutants from a variety of machinery used in various construction operations.
- Dust from excavations. The quantities of dust emissions from roads and non-asphalt surfaces range widely (1–10 kg/vehicle-kilometre) (Maropoulou, 2009).
- Additional emissions from the road traffic that serve the specific project.

19.3.3 Special features of each railway system category

According to

- A research conducted in 2006 by the Centre for Neighborhood Technologies, high-speed trains release into the atmosphere 30–70 gr CO_2/pkm (CNT, 2006).
- Data from DB AG TREMOD (Kettner, 2008), the emission of CO_2 in long-distance passenger railway transport is 49 gr/pkm.
- A study conducted in 2010 by JR Central in the Tokyo–Osaka connection, with a length of 515 km, the emissions of CO_2 for the N700 'Nozomi' train were estimated to be 8.54 gr/pkm (JR Central, 2010).

Figure 19.3 displays the emissions of CO_2 per passenger train category in Europe and for different traction systems.

By evaluating the above data, we can state that high-speed trains emit carbon dioxide (CO_2) in the range of 10–50 gr/pkm.

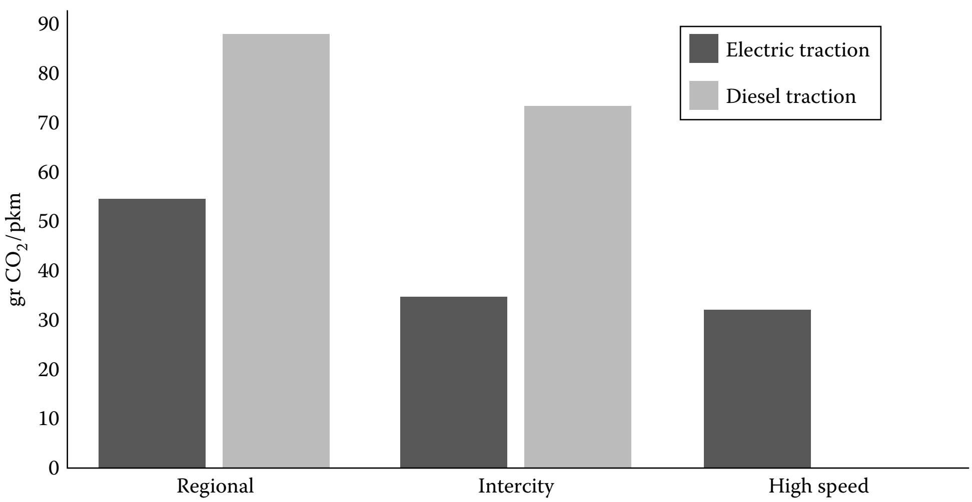

Figure 19.3 CO_2 emissions per train and traction type in Europe, 2005. (Adapted from UIC. 2012, *Energy Consumption and CO_2 Emissions, Railway Handbook*, 2012.)

Table 19.5 Range of fluctuation of the values of air pollution for various railway system categories

Interface with the environment	*High-speed trains*	*Conventional-speed trains*	*Freight transport (UIC, 2012)*
Air pollution (during system operation)	10–50 gr CO_2/pkm (30 gr CO_2/pkm)	Electrification 15–65 gr CO_2/pkm (40 gr CO_2/pkm) Diesel traction 75 gr CO_2/tkm	Electrification 18 gr CO_2/pkm Diesel traction 28 gr CO_2/pkm

The conventional-speed railway emits 29% more pollutants per passenger than the high-speed railway (Garcia, 2010b).

With regard to the tramway, the only ambient air pollutants that it may cause are the particles derived from the friction between wheels and rails; however, due to their small quantity, these are considered to be negligible, compared with other pollutants within a city. According to studies conducted in Melbourne, the emissions of carbon dioxide that correspond to each passenger-kilometre are estimated for tramway at an average of 60 gr CO_2/pkm.

Table 19.5 displays the range of fluctuation of the values of air pollution for various railway system categories.

19.3.4 Measures for air pollution reduction

The railway is the only means of transport whose share with regard to carbon dioxide (CO_2) emission has been reduced since 1990, while the respective share of all other transport means has increased. The total CO_2 emissions from railway operations in the EU were reduced by 39% from 1990 to 2010. More specifically, CO_2 emissions were reduced by 27% per passenger-kilometre and 41% per train-kilometre (UIC, 2012).

Beyond the application of electrification, in order to deal with air pollution, a variety of new techniques, constructions and environmentally friendly materials have been used, such as

- *Hydrogen trains*: The term ‘hydrogen train’ (hydrail) implies all types of railway vehicles, large or small, which use hydrogen as a source of energy to power their locomotives. These trains convert the chemical energy of hydrogen to mechanical energy.
- Natural gas, biofuels
- *Cement sleepers with the addition of slag*: Cement causes significant environmental impacts because of the CO_2 that is emitted during its production. The construction of this environmentally friendly sleeper has been performed by replacing part of the high-durability Portland cement with granulated slag (ground-granulated blast furnace slag), and part of the fine-grained aggregates with oxidising slag (electric-arc furnace oxidising slag).
- *Brakes with pads made of natural fibre*: Many railway companies want to abolish the use of porous metals in brakes, which results in the diffusion of heavy metal particles into the environment. According to a research by STI (Sustainable Technologies Initiative), natural fibres can replace the costly aramid fibres used for brake pads without any reduction in performance, and with clearly less impacts to the environment (reduction of dust contained in these materials, and which is released to the environment, as pads wear).

Figure 19.4 Solar panels on top of railway tunnel, Belgium. (Adapted from Gifford, J. 2011, *High-Speed Rail Line Installation Powers First Solar Train*, available online at: http://www.pv-magazine.com/news/details/beitrag/high-speed-rail-line-installation-powers-first-solar-train-_100003253/#axzz3Xy0A1P8R (accessed 30 April 2015).)

- *Renewable energy sources (solar, aeolic, and geothermal)*: An example of the use of solar energy is the railway tunnel (cut-and-cover tunnel) near Antwerp, Belgium, with a total length of 3.4 km, on top of which 17,820 solar panels have been installed, generating 3,300 MW h of electrical power every year (Figure 19.4). The Blackfriars railway station in London, which has been constructed on the bridge bearing the same name, hosts 4,400 photovoltaic panels on its roofs. The installation was expected to cover 50% of the station's electrical power requirements. In Japan, solar panels are installed on stations and facilities, while in Italy the installation of wind power generators on bridges is investigated (aeolic bridges) (Pikal, 2011).

19.4 SOIL AND WATER POLLUTION

19.4.1 Definition: Measurement methods of soil and water pollution

Soil pollution is defined as any unwanted change in the physical, chemical and biological properties of soil, which under conditions is, or may become, harmful to humans and other plant and animal organisms.

Soil and water pollution may be defined in the following ways:

- In the event of fuel leak, by identifying the soil content of insoluble hydrocarbons, polycyclic aromatic hydrocarbons and volatile aromatic compounds, which are toxic substances contained in oil
- By measuring the pH of soil and water
- By measuring the organic substances ending in underground and surface waters, using the BOD (BODu) index (biochemical oxygen demand)
- By measuring the concentration of foreign materials in the soil, in terms of mass per volume unit
- By measuring the concentration of foreign materials in water, in terms of mass per volume unit

19.4.2 Railway activities causing soil pollution

The soil pollution that is caused as a result from the presence of a railway system is due to the (Donta, 2010)

- Sealing of the soils (i.e. when the soil surface is covered by watertight material, e.g., concrete).
- Release of inorganic and organic substances to the environment by the passing trains. These substances may end into the underground and surface waters, after permeating the track bed layers and passing in the foundation soil.
- Combat of the vegetation on the track superstructure, by using pesticides that are hard to degrade.
- Disturbance of the flow of underground waters during the construction of the railway system.
- Pollution resulting from an accident, during the transportation and storage of fuel, hazardous goods and other materials, causing their leak into the ground and underground waters.
- Pollution because of liquid waste, emerging from the activities that take place in the area of the depot.

19.4.3 Special features of each railway system category

The problem of soil and water pollution involves mainly the operators of urban railway systems and systems of railway transportation of hazardous goods.

Specifically, tramway vehicles possess a liquid waste cleaning unit which is installed on the train itself, while they normally use a superstructure which is embedded in the pavement. The areas with a high likelihood of soil degradation are mainly the railway facilities/premises.

With regard to the railway transportation of hazardous goods, due to the nature of the products in case of an accident, leaks from vehicle tanks or from packages, or just poor handling by the staff during loading/unloading or transfer of materials, the risk of soil pollution is high. Further to the accident prevention measures that, apart from the safety, also guarantee the protection of the environment, there are also measures that aim exclusively at the protection of the soil, such as the following:

- Waste drainage system in the washing stations of empty tanks.
- Special configuration of the track superstructure (e.g.,slab track instead of ballasted track) in the siding lines of reception and forwarding of trains transporting hazardous goods.

The various activities associated with the movement of these products are regulated by international conventions and are performed under strictly defined safety conditions. For railway transportation the provisions of COTIF/CIM/RID apply (Europa, 2012).

19.4.4 Countermeasures against the pollution of soil due to the presence of the railway

The most significant countermeasures against the pollution of soil and waters due to the presence of the railway are the following:

- Regular monitoring of the quality of the soil and water
- Maintenance of facilities and fuel storage tanks

- Placement of special surface layers (covers) and drainage systems at places where the train stops, such as, before the lighting signals at the stations, and in warehousing and maintenance areas
- Placement of special oil collectors to prevent pollution from leaking oil
- Equipping vehicles with composting toilets

19.5 VISUAL ANNOYANCE

19.5.1 Definition: Measurement methods of visual annoyance

Visual annoyance is divided into visual obstruction and visual nuisance.

Visual obstruction is defined as the rate of an observer's field of vision that is covered by the infrastructure and the facilities/premises of the railway system. It is estimated through the angle formed between the point of observation and the relevant railway structure.

Visual nuisance is defined as the discomfort caused to the observer by the presence of the infrastructure of the railway system. It can be assessed by questionnaire-based research, addressed to the users of the means of transport, and mainly to the citizens inhabiting or working close to the railway network.

In interurban railway networks, with regard to visual annoyance, more considerations are involved, which describe the level of attraction of the landscape and, to a lesser extent, the observer's visual nuisance.

The aesthetics of the civil engineering structures and the trains, as well as the level of attraction of the landscape, before and after the project constitute elements that can be graded subjectively by experts and by the public.

In urban railway networks, visual annoyance depends on the quantity of the field of view that is lost, due to the presence of the railway system.

In any case, the correct estimate of the visual outcome of a railway project can be performed only after the completion of its construction. Environmental design can provide for (or prevent) the potential visual annoyance, based on the existing experience in relevant projects.

19.5.2 Railway activities causing visual annoyance

Visual annoyance is caused during construction, as well as during operation of a railway system.

In the case of construction of the railway infrastructure of urban railway systems (metro and tram), strong visual annoyance is caused by the facilities of the construction sites and the fencing.

During the operation of railway systems, visual annoyance is caused by

- Electrification system installations (overhead catenary system, substations, and electrification masts)
- Signalling system installations
- Noise barriers
- The presence of high embankments
- Depots, when they are constructed in urban areas

19.5.3 Special features of each railway system category

The main parameters of visual annoyance caused by the tramway are the overhead catenary system. The aesthetic annoyance caused by the wires is significant when the tramway lines are integrated in highly populated and visited areas, and particularly in sensitive regions (historic centres, traditional residential settlements, monuments, etc.).

Another parameter of visual annoyance caused by the tramway is the depot, because it occupies a large ground plan area and is usually constructed within an urban area.

In general, however, the tramway's profile is particularly attractive to the citizens, while in many cases the tram itself comprises an attraction to the city.

The metro usually does not affect the aesthetics of cities, except only during the construction stage, due to the fact that it operates underground. With regard to underground railway stations, in many cities they are exceptionally decorated, and their aesthetics create a pleasant atmosphere for the user.

19.5.4 Countermeasures against visual annoyance caused by the presence of the railway

During the construction of any railway system, architectural design should constitute an integral part of the total design. Its goal should be the construction of facilities that are of high aesthetics while, at the same time, functional, and the delivery of user-friendly railway systems.

For the tram, the most effective solution to the aesthetic annoyance caused by the aerial cables is to perform power supply through the ground or via energy-storage devices (see Section 4.2.4 and Chapter 20), at least for those segments of the network that are environmentally sensitive. Other measures used in tramway networks for the reduction of visual annoyance are

- Covering the tram traffic corridor with grass (Figure 19.5). This solution, on top of the aesthetic enhancement of the landscape, prevents the effect of temperature increase in city centres, and provides a permeable layer that filters the precipitation waters, thereby limiting the pollution of the subsoil
- The aesthetics of trains and stops

Figure 19.5 Covering a tram corridor with grass, Montpellier, France. (Photo: A. Klonos.)

In interurban railway systems the usual measures used toward the reduction of visual annoyance are

- Planting of embankment slopes
- Fencing with plants
- Transparent noise barriers and the noise barriers with planting
- The aesthetics of the civil engineering structures

19.6 INTEGRATION OF THE TRACK INTO THE LANDSCAPE

19.6.1 Definition: Measurement indices of integration

A railway line is considered to be optimised in terms of the track alignment, when it consists exclusively of straight sections, and lies throughout its length on a horizontal plane. The integration of such an optimised track layout into the landscape requires earthworks and civil engineering structures, which alter the topographic relief and increase the implementation cost. The adoption of curved sections in the horizontal alignment and the adoption of longitudinal slopes in the vertical alignment reduce these interventions and smooth out the integration into the landscape.

The quality of integration of the railway network infrastructure is characterised by

- An excess of the alterations performed in the topographic relief
- Alteration in the drainage of surface waters
- The level of reinstatement of the landscape
- The land take

Earthworks, and specifically, embankments and cuttings, constitute interventions that alter the topographic relief.

The indices usually used for the estimation and evaluation of the impacts of the earthworks of a railway system are the height of the embankments, and the depth of the cuttings, as well as the volume of the soil that is removed or deposited for the requirements of the railway civil engineering structures.

The relatively small right-of-way (land take) of the railway track causes a much lesser problem to the natural environment than the motorway, resulting in the requirement for lower-width cuttings and embankments.

The reinstatement of landscape to its original form, or finding an accepted aesthetic solution for the new situation formed, is also of particular importance. The issue of the restoration of landscape has begun to gain significance during the last 30 years. Currently, the budget of the projects provides for 1%–2% to be spent on the reinstatement of the landscape.

19.6.2 Railway activities causing a change of landscape

The change of ground relief is mostly due to the execution of earthworks, and specifically due to the construction of embankments and cuttings.

Embankments and cuttings are constructed in order to ensure a smooth track alignment (straight sections where possible and small longitudinal slopes) in areas of difficult landscape.

Embankments are used as an alternative to bridges, for access to areas of high altitude difference, provided that their height is not prohibitive. They are also used at low altitudes, for the protection of the track superstructure from flooding.

Cuttings are used as an alternative to tunnels, for track layout within mountains provided that their height is not prohibitive.

Deep cuttings may cause landslides on the slopes, while high embankments may result in the blocking of access to certain areas.

19.6.3 Special features of each railway system category

Regardless of the category of railway system, the height of embankments should be, for reasons of traffic safety, relatively low, while the soil should have been compacted well (see Sections 18.5.5 and 18.5.6). Indicatively, it is stated that embankment heights of more than 20 m should be avoided.

The application of high speeds also involves the adoption of large curvature radii in the horizontal alignment, and a greater distance between track centres. This leads to a more difficult integration into the topographic relief.

The integration of the metro systems depends on the selection of the construction depth of the stations (see Section 5.4.2). The typical depth is 18–20 m from ground level; however, this fluctuates depending on the conditions.

19.6.4 Measures for smooth integration of the railway into the landscape

Some of the measures used for smooth integration of the railway systems into the topographic relief of an area are

- The selection of tunnel boring machines after evaluating the geologic and geotechnical conditions, in order to minimise ground surface settlement
- The construction of tunnels, rather than deep and long cuttings
- The construction of bridges, rather than high and long embankments (Figure 19.6)
- The reinstatement of landscape

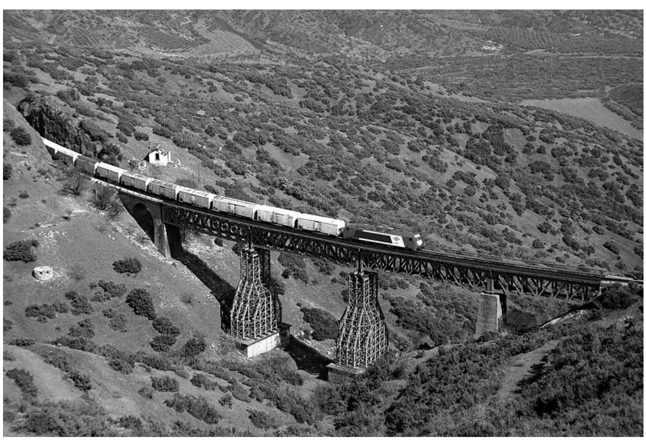

Figure 19.6 Bridge construction for smoother integration into the topographic relief, Karya, Greece. (Photo: A. Klonos.)

19.7 ECOSYSTEM DISTURBANCE

19.7.1 Definition: Indices of expression of ecosystem disturbance

The integration of a railway line has a direct impact on the biological resources of the area through which the line passes:

- Alteration or destruction of the natural environment and the reproduction areas of rare or endangered species.
- Loss of a significant number of species. Endangered species under higher risk are large mammals (deer, boars, and bears), migratory and prey bird species and bats.
- Impacts or measurable degradation of the protected natural environment, the sensitive natural vegetation, the wetlands and other environmentally protected areas.
- Conflict with the provisions of the existing environmental regulations on a local, regional or national level.

The restriction of access to the area can be assessed based on the impact observed on the fauna of the area (decrease or increase in the number of specific species, and, in general, the impact on the lifecycle of endemic species in the area, before the construction and during the operation of the railway network).

19.7.2 Railway activities causing ecosystem disturbance

Ecosystem disturbance is caused by

- The land take by the railway
- The track fencing
- The noise emitted by the railway

The compulsory fencing of main lines in high-speed networks constitutes an important protection measure toward the reduction of animal mortality rate, and in many cases it may act negatively by reducing their likelihood of survival, such as in the case that railway track crossing from one side to the other has become impossible.

19.7.3 Special features of each railway system category

The problem relates mostly to high-speed networks, to which fencing is imposed, and in general to all tracks passing through habitats.

19.7.4 Reduction measures of ecosystem disturbance

The most effective measure toward the reduction of ecosystem disturbance is the correct selection of the track alignment in the first place, and specifically the avoidance of passing of the railway track through areas characterised as sensitive.

The mitigation measures used for the reduction of ecosystem disturbance are:

- Ensuring the continuity of natural habitats by using wildlife crossings. Wildlife crossings may include underpass tunnels, viaducts and overpasses (green bridges); amphibian tunnels, fish adders, tunnels and culverts; green roofs (http://en.wikipedia.org/wiki/Wildlife_crossing, 2015) (Figures 19.7 and 19.8).

Figure 19.7 Wildlife overpass on a railway track, TGV-Atlantique, Lavare, France. (Adapted from Vignal, B. 1990, SNCF Médiathèque.)

The factors that define the effectiveness of overpasses for animals are

- Proper vegetation in their entrance points
- Their dimensions, the presence of natural lighting and the level of noise
- Their proper location
- Systematic monitoring of animal movements and behaviour
- The exchange of knowledge and cooperation among researcher engineers, biologists and ecologists

The monitoring of overpasses reveals that animals become familiar with them and use them. Nowadays, the number of this specific type of overpass in Europe is fairly high, while in the last years it has been attempted to increase their number in America:

Figure 19.8 Wildlife overpass on a railway track, TGV-Atlantique, France. (Adapted from D'Angelo, J.J. 1995, SNCF Médiathèque.)

- Special protection of endangered flora, implantations of trees and shrubs and special reforestation campaigns.
- Emission of sounds before the passage of trains in order to move animals away from the track. Moving dead animals that attract birds, away from the tracks.
- The prevention of forest fires.
- Use of a camera on the train assisted by a computer program in order to detect weeds along the track.

19.8 DISTURBANCE OF LOCAL RESIDENT ACTIVITIES: ACCESS RESTRICTION AND DISRUPTION OF URBAN SPACE

19.8.1 Definition: Measurement indices of disturbance on local resident activities

The permanent way of a railway system cuts off and isolates the area from where it passes. The isolation caused has impacts on the communication and movement of humans on either side of the railway track, resulting in the disturbance of several of their activities.

When a line passes through urban areas, the continuity of the urban space is disrupted. This results in a reduction in the accessibility of various destinations which may be of interest to both pedestrians and road vehicles.

In case of passage of the railway infrastructure through non-urban areas, the agricultural and livestock farming activities are obstructed. With regard to agricultural farming, the operation is literally split into two pieces. This constitutes an important issue, and there are many examples of cases where it has not been solved effectively. It usually comprises the main cause of discord between railway organisations and farmers.

The right-of-way, the length of the line, the segregation level (e.g., fencing and insufficient side road network) and the characteristics of the area themselves determine the volume of disturbance caused to the activities of local residents.

19.8.2 Railway activities causing disturbance on local resident activities

The disturbance of activities resulting from the presence of the railway is due to the fact that the railway uses an exclusive traffic corridor. The track fencing imposed on high-speed networks, as well as on conventional-speed networks which, for part of their length are integrated into urban and suburban areas, intensifies the problem.

Accessibility problems are also caused during the construction of urban railway systems.

19.8.3 Special features of each railway system category

With regard to the metro, there is no disruption arising in the urban space because the metro moves under the ground. Its stations in fact increase, to a great extent, access to nearby areas, permitting urban regeneration. At the same time, concentrated development around the stations reverses the spatial dispersion of activities caused by the use of private cars.

The tramway, being a means of transport that moves at grade, is characterised by the permanent occupancy of a specific part of the road's width, and the necessity to receive priority against other means of transport at intersections. Therefore, the traffic capacity of the road is reduced, while the delay of road vehicles at intersections is increased. Its impacts on the various activities and access are directly linked to the category of the corridor used, and the way it integrates along the width of the road arteries.

19.8.4 Measures for the reduction of disturbance caused to local residential activities due to the presence of railway infrastructure

The maintenance of the residential structure of the surrounding area and the protection of the quality of life, mostly for the population located within a small distance from the track (0–500 m), constitute a primary concern toward the preservation of the cultural environment. Furthermore, the maintenance of agricultural activity in areas of high sensitivity, in terms of agricultural use, and of course the maintenance of continuity of archaeological sites, whether officially declared as such or not, are the main features that are taken into account in the process of the track alignment.

The measures used in order to reduce the disturbance of activities caused by the passage of a railway track are

- *The construction of overpasses/underpasses at sufficient distance*: On the basis of the type or the functionality of the transport means which intersect at different levels, unlevelled crossings are divided into the following categories:
  - Railway–railway
  - Railway–road
  - Railway–pedestrian bridge (Figure 19.9)
  - Railway–animal passage (Figures 19.7, 19.8 and 19.10)

  Railway overpasses/underpasses have to be constructed at a specific location along the line, based on traffic flow and environmental study. In each case, it is required to be provided for the rehabilitation of the local road network, in order for movement and communication on either side of the track to run smoothly, where possible.
- *The construction of level crossings at sufficient distance:* Level crossings are part of track side systems and constitute a basic structural and functional component of all conventional-speed railway networks ($V < 200$ km/h).
- Finally, during the construction of urban railway networks, and specifically of underground railway stations, traffic arrangements are required.

Figure 19.9 Pedestrian bridge, N. Poroi, Greece. (Photo: A. Klonos.)

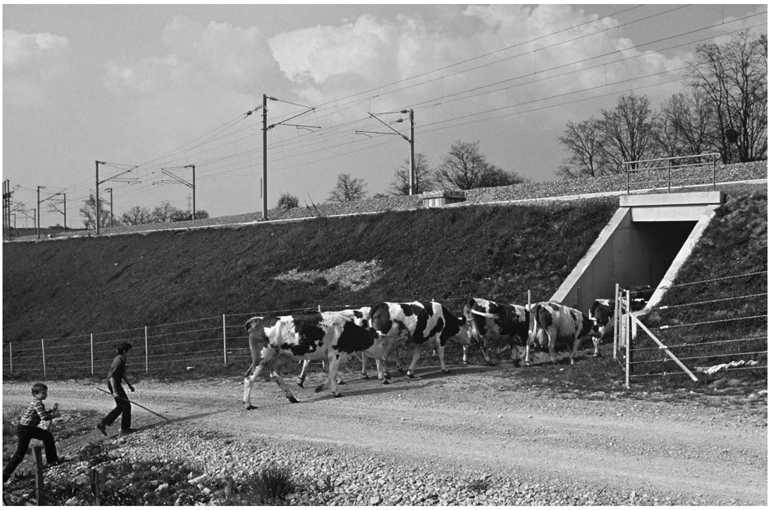

Figure 19.10 Animal underpass, TGV Sud-Est, Saint-Laurent-d'Andenay, France. (Adapted from Henri, M. 1980, SNCF Médiathèque.)

19.9 ACOUSTIC ANNOYANCE

19.9.1 Definition: Units of expression of acoustic annoyance

Acoustic annoyance is proportional to the acoustic energy received by the human ear. Acoustic energy is a function of the noise level, and the duration of exposure to that noise.

Noise could be defined in two ways, as given below:

- Any irregular, non-periodic noise, the instant value of which fluctuates in a random way
- Any unwanted noise

Noise is directly linked to human health and well-being.

The energy indices commonly used for the assessment of railway noise, and its impacts on humans, are the equivalent energy noise level $L_{eq,T}$, the index L_{dn} (day–night equivalent noise level), the index L_{den} (day–evening–night equivalent noise level), the sound exposure level (SEL) and the maximum noise level L_{max}. All these indices are measured in dB (A) (http://www.eea.europa.eu/data-and-maps/indicators/traffic-noise-exposure-and-annoyance/noise-term, 2001; Tsohos et al., 2001; UIC, 2010).

Table 19.6 displays the permissible noise levels for humans.

Table 19.6 Permissible noise levels for humans

Noise level (dB (A))	*Annoyance description*
≥81	Unacceptable situation
80–78	Very noisy situation
77–75	Noisy situation
74–72	Nearly bearable situation
71–69	Good situation
≤68	Comfortable situation

19.9.2 Railway activities causing acoustic annoyance

Depending on their source, five types of noise are distinguished during the operation of railway transport. These are:

Mechanical noise caused by the electromechanical equipment of the train, and primarily the power vehicles. It originates from various sources, which are indicatively the power vehicle motors, the braking system, air compressors, the air conditioning, etc. This type of noise is perceived and gains more importance, mostly during movement at low speeds (30–40 km/h).

Rolling noise (Kitagawa, 2009) caused by the 'wheel–rail' system. This noise is generated during the contact of wheels with rails, and, secondarily, by car body oscillations. The causes for the generation of this noise are indicatively the discontinuities of the track (e.g., the presence of rail joints, switches and crossings), the wheel flange–rail contact, the hunting of the railway wheelset due to geometric track defects, etc. (Schweizer Norm. SN 671 250a, 2002). In general, the rolling noise plays a significant role in medium speeds, where it comprises the basic source of noise. The rolling noise generated during the contact between wheel flange–inner rail edge (negotiation of bogies in curved horizontal alignment sections of very small radius as in metro, tram and depot tracks) is particularly annoying, and is called squeal noise.

Aerodynamic noise, which is prevalent at very high speeds ($V > 250$ km/h), is perceptible at high speeds (200 km/h $< V <$ 250 km/h), while it is considered negligible at conventional speeds ($V < 160$ km/h). It is due to the increase of aerodynamic resistances during the movement of the train; it depends significantly on the aerodynamic shape of the head and the rear of the train, its cross-sectional surface, the condition of its lateral surface, its length, the number of vehicle bogies, the aerodynamic protection of pantographs, etc.

Arcing noise, which is caused by electrical traction, is generated in the event of discontinuity in the contact of pantographs with the aerial power supply cables. This noise is very similar to that generated by trolleybuses.

Ground-borne noise, which is perceived by the occupants of the buildings near the railway track as a very low-frequency noise. It originates from the vibrations caused during the movement of the railway vehicles, either in a surface track or in an underground track (inside a tunnel), and which are transferred through the ground to adjacent buildings. Ground-borne noise is examined in Section 19.10.

Elevated structure noise, which is attributed to railway systems operating on elevated guideway.

Finally, *construction site noise*, arising during the construction stage of a railway project, and generated by the construction site machinery.

19.9.3 Special features of each category of railway systems

Acoustic annoyance concerns all railway systems. Table 19.7 displays the correlation between the type of noise, as defined in Section 19.9.2, and the various categories of railway systems.

The noise emitted during the passage of a high-speed train increases by up to 300 km/h as a function of the third power of speed, while for speeds higher than 300 km/h, it increases as a function of the sixth to eighth power of speed (Relié, 1989; Maeda et al., 2010).

Noise limits in the high-speed railway are clearly defined in the technical specifications for interoperability of the trans-European high-speed railway system (EC, 2008). According to these specifications, the external noise emitted by the rolling stock of the high-speed railway system is divided into noise at stop, startup noise and rolling noise.

Table 19.7 Correlation between the type of noise and the various categories of railway systems

	High-speed railway	*Conventional-speed railway*	*Metro*	*Tramway*
Mechanical noise	+	+	+	+
Rolling noise	+	+++	+++	+++
Squeal noise		+	++	+++
Aerodynamic noise	+++	+		
Arcing noise	+++	+ (electrification)	++	++
Ground-borne noise	+	+	+++	++
Construction site noise	+	+	+++	+++

+: Correlation.
++: Strong correlation.
+++: Very strong correlation.

Table 19.8 Measurement results of rolling noise of high-speed trains in Europe (UIC)

Passage speed (km/h)	*Rolling noise values measured at a distance of 25 m (dB (A))*
250	86–90
300	89–92
320	91–93
350	94.5–97

Source: Adapted from Eadie, D.T., Kalousekb, J. and Chiddicka, K.C. 2002, *Wear*, 253(1–2), 185–192.

Table 19.8 displays the measurement results of high-speed trains' rolling noise in Europe (UIC).

An increase in the roughness of wheels may induce an increase of noise by 3–4 dB (A) (Eadie et al., 2002).

The chart of Figure 19.11 displays the relationship between the running speed of the tram and the noise caused.

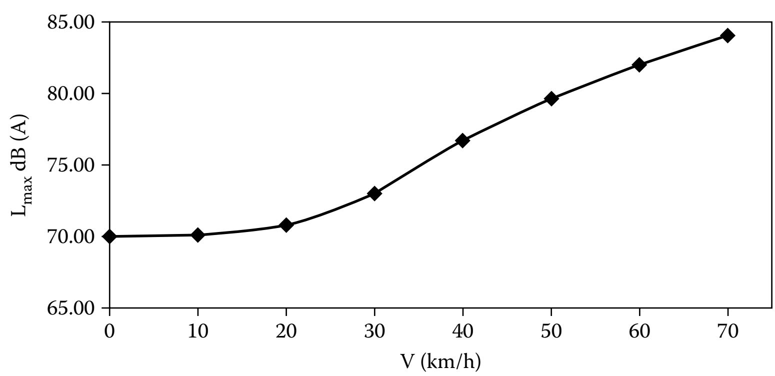

Figure 19.11 Relationship between external noise of the tram and running speed. (Adapted from Oikonomidis, D., Triantafyllopoulos, P. and Paidousi, M. 2003, Noise protection programme during the operation of the tram of Athens, *International Conference Contemporary Tram and LRT Systems*, 19–20 May 2003, Patras.)

Figure 19.12 Noise barriers made of reinforced concrete, TGV ATLANTIQUE, France. (Adapted from Olivain, P. 1989, SNCF Médiathèque.)

Finally, with regard to freight trains, according to Kurer, a typical train formation emits 90 dB (A) (ECMT, 1993).

19.9.4 Countermeasures against acoustic annoyance

For the reduction of acoustic annoyance caused by railways, countermeasures are taken along the path of noise transmission and at the source of cause of noise. Almost all measures used at the source of cause of noise are also used toward the reduction of ground-borne vibrations and noise (see Section 19.10).

19.9.4.1 The path of noise transmission

The measures used toward the reduction of railway noise along the path of its transmission are noise barriers (Figure 19.12) and fencing with planting (Figure 19.13). These measures (overall noise reduction potential 5-15dB, UIC, 2010) are effective only on a local level, since for the protection of longer sections of the railway networks, high investment is required.

Figure 19.13 Noise barriers made of concrete with planting. (Adapted from Online image, available from: http://www.tucrail.be/FR/media/Site_of_the_month/PublishingImages/2013/chantier_du_mois_06_2013_3.jpg (accessed 30 April 2015).)

Table 19.9 Values of noise-reduction index R_s for various frequencies

Frequency (Hz)	100	125	160	200	250	315	400	500	630
R_s (dB)	10	12	14	16	18	20	22	24	26
Frequency (Hz)	800	1,000	1,250	1,600	2,000	2,500	3,150	4,000	5,000
R_s (dB)	28	30	31	32	33	34	35	36	36

Source: Adapted from Schweizer Norm. SN 671 250a. 2002, *Schweizerischer Verband der Strassen – und Verkehrsfachleute* (VSS), May 2002.

The level of acoustic annoyance for the railway is higher compared with that of a road network; therefore, the construction material of the noise barriers should be characterised by higher sound-insulation performance, compared with that of a road network. Specifically, pursuant to regulation SN 671 250a, the noise barriers in the railway should ensure the reduction of the railway noise, at least by 10–15 dB.

Table 19.9 displays the values of the noise-reduction index R_s (an index which expresses the sound-insulating capacity of the construction material of noise barriers, having dB as a unit of measure), for specific frequencies.

To improve the efficiency of noise barriers on both sides, and for a height ≥2 m, adequate sound absorption should be ensured. Sound absorption, measured by coefficient α_s (sound-absorption coefficient), should, at least and at specific frequencies, take the values that are displayed in Table 19.10.

The maximum height distance between the noise barriers and the rolling surface of the rails is commonly 2 m, in order for the landscape aesthetics as well as the attractiveness of the adjacent inhabited areas and the visibility of the train passengers to be preserved (Bontinck, 1997).

When the height of 2 m is not sufficient for adequate sound protection from railway noise, noise insulated windows are placed in the affected buildings (overall noise reduction potential 10-30dB, UIC, 2010). The placement of even higher noise barriers is selected only when there are serious reasons.

Low noise barriers at close distance from the tracks are avoided due to bad acoustic and low safety level. However, they are the last resort, in case there is no other alternative.

19.9.4.2 The source of noise

The measures taken at the source reduce the noise throughout the railway system, provided that they are applied widely. Such measures are

- *The use of brakes made of composite materials*: The replacement of wheel pads made of cast iron, by wheel pads made of composite material (retrofitting with K-blocks or LL-brake blocks), ensures that the rolling surfaces of the wheels remain smooth; this results to a reduction of the rolling noise of up to 10 dB (UIC, 2010).

Table 19.10 Values of sound-absorption coefficient α_s for specific frequencies

Frequency (Hz)	100	125	160	200	250	315	400	500	630
α_s	0.10	0.15	0.20	0.25	0.30	0.45	0.60	0.75	0.80
Frequency (Hz)	800	1,000	1,250	1,600	2,000	2,500	3,150	4,000	5,000
α_s	0.85	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90

Source: Adapted from Schweizer Norm. SN 671 250a. 2002, *Schweizerischer Verband der Strassen – und Verkehrsfachleute* (VSS), May 2002.

- *The use of resilient wheels (Figure 19.14)*: Thanks to their special configuration, they reduce the ground induced vibration through the reduction of unsprung and the noise through increased wheel damping. An additional beneficial effect is that resilient wheels absorb more roughness excitation than a monobloc wheel (because of the lower radial stiffness) and thus reduce the noise radiated by the rail. It has been observed that by using resilient wheels, the noise level is reduced by 5–6 dB. This technique is used mostly in tram vehicles (obviously resilient wheel could reduce even 10–15 dB of squealing noise).
- *Rail vibration dampers*
- *The use of exclusively continuous welded rails and elastic fastenings*
- *The aerodynamic design of pantographs*
- *The absence of aerial wires*
- *The proper maintenance of the rolling stock (reprofiling of wheels with wheel flats) and of the track (rail roughness can be kept low through acoustic grinding)*
- *Wheel dampers (Betgen et al., 2012)*

19.10 GROUND-BORNE NOISE AND VIBRATIONS

19.10.1 Definition: Measurement units of ground-borne noise and vibrations

One of the most important environmental impacts of railway systems, and specifically of urban railway systems moving on underground track sections, are the vibrations caused by the train movement which are borne and transmitted through the ground to its surface and the overlooking adjacent buildings (Figure 19.15).

The ground-borne vibrations are perceptible by the residents through the vibrations of the floors and the walls of the buildings. They are annoying, since they can cause not only the movement of various objects inside the buildings, but also a secondary noise which originates mostly from screeching of glass panels and cooking utensils. This noise

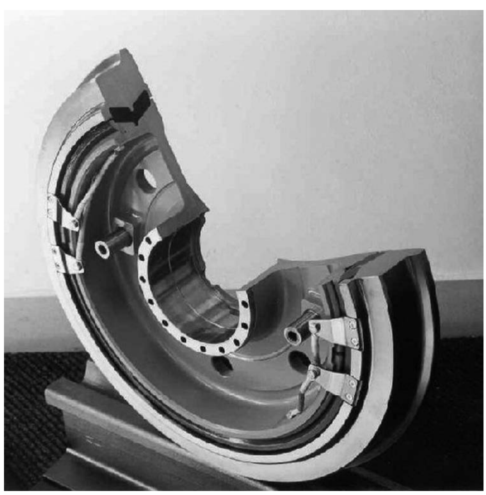

Figure 19.14 Resilient wheels. (From BONATRANS GROUP, A.S., 2015.)

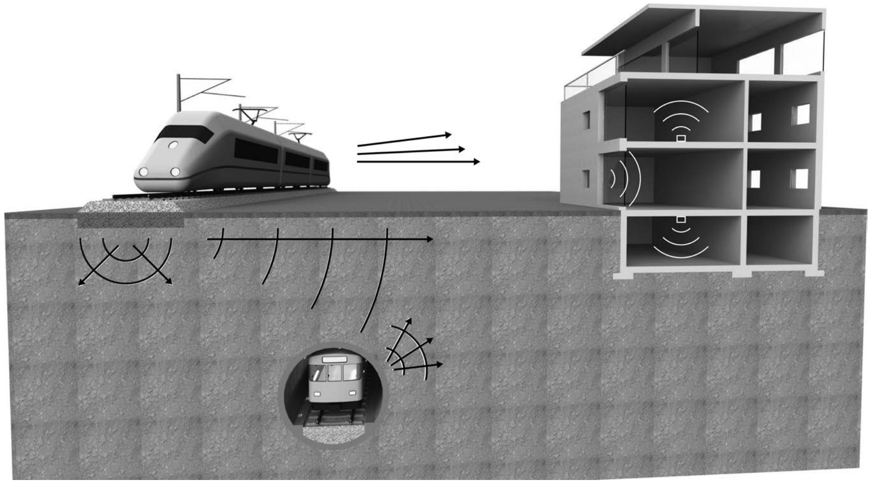

Figure 19.15 Transmission of vibrations in the area around railway lines. (Adapted from Loy, H. 2014, available online at: http://innorail.hu/en/vibration-mitigation-with-under-sleeper-pads/)

(ground-borne noise) is caused by vibrations in frequencies usually in the range of 40–80 Hz, because of the resonance of the building constructions in the tunnel vicinity. It becomes perceptible as a low-level muffled noise (rumble).

Given that the minimum typical limit of the human acoustic capacity is about 20 Hz, vibrations under this frequency become perceptible by the residents of the buildings as vibrations (ground-borne vibrations), while over this frequency they become perceptible both as vibrations and as sound.

Inside the railway vehicles, vibrations reduce the vertical dynamic comfort of passengers. The size of the vibrations can measured based on the

- Displacement d_o (mm)
- Velocity v_o of the vibrations (mm/s)
- Acceleration α_o of the vibrations (mm/s^2)

Vibrations are primarily expressed in the form of velocity or acceleration. They can, however, be expressed also in the form of noise levels, in dB.

The following conversion relationships apply:

$$L_{\alpha o(\mathrm{level})} = 20\log_{10}(\alpha_o/\alpha'_o)\ (\mathrm{dB}) \tag{19.1}$$

where

α_o: acceleration of vibration in m/s^2

α'_o: reference level of the acceleration of vibration, 10^{-6} m/s^2 or 10^{-5} m/s^2

The product $20\log_{10}(\alpha_o/\alpha'_o)$ (dB) is defined as VAL (vibration acceleration level)

$$L_{v(\mathrm{level})} = 20\log_{10}(v_o/v'_o)\ (\mathrm{dB}) \tag{19.2}$$

where

v_o: velocity of vibration

v'_o: reference level of the velocity of vibration, 10^{-9} m/s

The product $20\log_{10}(v_o/v'_o)$ (dB) is defined as VVL (vibration velocity level)

$$L_{d(level)} = 20\log_{10}(d_o/d'_o) \text{ (dB)} \tag{19.3}$$

where

d_o: displacement

d'_o: reference displacement, 10^{-11} m

An index used for the description of the amplitude of vibrations (as well as sound waves) is the root mean square (rms) of vibration velocity or acceleration (rms). It results from the summary of the square values of velocity or acceleration at each instant, and the calculation of the mean duration of a period. The rms of velocity or acceleration is the square root of this mean value of time.

The impacts of vibrations on damage caused to the constructions are estimated through comparisons based on peak particle velocity (PPV). In general, the vibration levels required to cause damage to the buildings are much higher than those considered acceptable by humans themselves (dynamic comfort).

The basic safety limit for avoiding damage is that the PPV value should not exceed 50 mm/s on the ground surface. The total of major damage to the buildings, as well as 94% of smaller damage, has been observed at PPV > 50 mm/s. Since the above limit does not fully guarantee that damage (major or minor) will not occur, Table 19.11 displays more conservative limits.

Table 19.12 displays the permissible levels of ground-borne noise for various land use and building types.

Table 19.11 Recommended maximum vibration velocity limits during construction

	Recommended vibration limits	
	Weighted acceleration (mm/s²)	*Equivalent velocity (mm/s)*
Other buildings	0.5–1	13–28
Special-use buildings and monuments	0.05	1.3

Table 19.12 Permissible levels of ground-borne noise

	Ground-borne noise (dB (A))		
Space type	*Detached house*	*Block of flats*	*Hotel*
Low population density	30	35	40
Average residential density	35	40	45
High residential density	35	40	45
Commerce	40	45	50
Industry–interurban network	40	45	55

Source: Adapted from Vogiatzis, K. 2011, *Information Bulleting of the Hellenic Association of Transportation Engineers*, Issue No 176, April–May–June 2011.

The vibration impacts to humans are assessed based on the rms of velocity. In general, at low frequencies (1–10 Hz) disturbance is due to acceleration, while at high frequencies (10–100 Hz), it is due to velocity (<7 mm/s).

In general, the range of vibration frequencies for an underground railway system fluctuates between 0 and 200 Hz (Eisenmann, 1994; Esveld, 2001; Lichtberger, 2005). The range of vibration frequencies that are of interest inside tunnels is 0–500 Hz, while for the ground surface, and therefore for the buildings, is 1–80 Hz. Vibrations in buildings usually peak at 50–63 Hz.

The critical condition emerges when the primary vibration frequency of the ground coincides with the natural frequency of the building. While the primary transversal vibration frequency of the building is 1–10 Hz, its individual structural elements can have higher natural frequencies.

The critical distance between tunnels and buildings, for ground-borne vibrations and for a ballasted track, fluctuates between 15 and 25 m (Eisenmann, 1994).

Ground-borne noise in buildings should be measured in the middle of a room. The ground-borne and the secondary noise (rattling) should not be included in the measurements. It is expressed by the sound pressure level (SPL), and is measured in dB (A) (mathematical expression 19.4) (Eitzenberger, 2008). The frequency range that is of interest is between 16 and 250 Hz:

$$L_p = 20\log_{10}(p_o/p'_o)\ (\mathrm{dB}) \tag{19.4}$$

where

p_o: the mean noise pressure in N/m^2

p'_o: the relative mean reference pressure, usually equal to 2×10^{-5} N/m^2 for transmission through the air and 0.1 N/m^2 for any other means of transmission

The ground-borne noise can be estimated if the ground-borne vibrations are known.

19.10.2 Railway activities causing and affecting ground-borne noise and vibrations

Ground-borne noise and vibrations are caused essentially during train movement at constant speed and acceleration.

The parameters that affect the transmission of vibrations and the noise caused through them are

In terms of the source of cause (wheel–rail interface, track superstructure, and rolling stock) (Nelson, 1996; Cox, 2002; Busch et al., 2005; Fujii et al., 2005)

- *The train running speed*: For speed between 25 and 115 km/h, an increase in the speed by 100% results in an increase of index VAL by 4–6 dB.
- *The train axle load*: Doubling the axle load, results, on tunnel level, in an increase of index VAL by 2–4 dB, in a frequency region between 40 and 250 Hz.
- The sleepers.
- *The train type and length*: Trains with vehicles of uniform load cause higher vibration levels.
- *The ballast thickness (in case of ballasted track)*: A fluctuation of ballast thickness from 30 to 70 cm under the sleeper does not have any effect.

  A slab track, compared with a ballasted track, has a higher noise level. Specifically, it reflects the rolling noise instead of absorbing it. It is estimated that the level of the emitted noise in the case of a slab track (without resilient fastenings and elastic pads) is 10 dB (A) higher than that of a ballasted track.

- *The wheel–rail contact surface quality (conditions and quality of rolling)*: The roughness of wheel–rail comprises the major factor that stimulates the rolling noise. Increasing the roughness alone results in an increase of index VAL by 3–10 dB.

  According to measurements performed in Germany, France and Japan, eliminating the wave corrugations of the rails by grinding may result in the reduction of the rolling noise by 6–14 dB, within a frequency range of 500–2,000 Hz. Measurements performed in the metro of Naples have shown that eliminating the wave corrugations at frequencies between 0.8 and 80 Hz results in a reduction of index VAL on the level of tunnel walls by 9 dB, while at frequencies between 20 and 250 Hz, by 17 dB.

  The elimination of wheel flats may lead, as recorded by measurements, to noise reduction by 3 dB (A).

  Bad rolling conditions in total (loose fastenings, wave corrugations of rails, etc.) may cause an increase of index VAL by 10–20 dB.

  The presence of continuous welded rails (CWR) reduces the vibration levels, as it eliminates discontinuities on the rolling surface. An increase has been recorded of index VAL by 5 dB, compared with the jointed track.
- *The braking system of the vehicles*: Vehicles with disc brakes cause less noise than those with cast-iron pads.
- The stiffness of the primary suspension of vehicles, and their dynamic behaviour in general, as well as the parameters affecting it.

  The low values of vertical stiffness of the primary vehicle suspension system reduce vibrations. Indicatively, it is stated that in the event of vehicles with a stiff suspension, vibration increases were measured, up to 10–15 dB.

In terms of the path (tunnel walls, soil, and earthworks)

- *Construction of the tunnel's bearing structure*: Doubling the width of the tunnel ring results in the reduction of index VAL (inside the tunnel) by 5–18 dB.

  The vibration levels inside the tunnel, in case the tunnel is constructed within rock, are higher by 12 dB (in high, audible frequencies), and lower by 5 dB (in low frequencies), than in the case where the tunnel is not constructed within a rock.

  The cut-and-cover sections (with a large cross section and close to the ground surface) generate higher noise levels than the sections constructed in tunnels with circular cross section.

In terms of the final receptor (buildings and humans)

- *The foundation system of buildings*: In case of foundation with slabs on grade, the reduction for frequencies higher than the natural frequency of the concrete slab is zero. The same applies to light constructions and buildings founded directly on a rock.

  In case of the foundation of a building on pillars, the vibration levels are higher down to the ground, close to the pillars, than on the ground surface.

  For other cases of foundation, there is a reduction by 2–15 dB, depending on the foundation system and frequency of vibration.
- *The distance of the building from the tunnel*: Space waves attenuate by a rate of 6 dB (50% in amplitude) as the distance of the source of vibration doubles, without the presence of a damping material in the ground.

- *The depth of the tunnel*: Underground constructions close to the ground surface generate higher vibration levels, than deep underground constructions.
- *The properties of the soil and the rock mass*: The velocity of wave propagation is higher in a dense and solid rock, and lower in a less dense and solid one. In general, in wetlands with muddy soils the vibrations attenuate fast, in sandy soils less fast, while in contrast, high vibrations are observed in hard soils near the track.
- *The construction material of the building's frame*: The generated vibrations affect more the constructions that are made of light wood and of metal.
- The number of floors of the buildings

The acceleration level of vibrations is reduced as we ascend to upper floors of a building. This attenuation fluctuates around 3 dB, and is higher at high frequencies.

19.10.3 Special features of each category of railway systems

Ground vibrations caused by the passage of a high-speed train are similar to those caused by the passage of a conventional-speed train. The vibration level, however, is relatively lower, due to the high standards observed during the construction, operation and maintenance of a high-speed railway track (Nelson, 1996).

The values of vibration of high-speed trains (V = 240 km/h) and in a distance of 30.5 m were estimated in the range of 75–80 V dB (VVL).

In trams, the accumulation of debris on the track superstructure surface causes vibrations of a fairly high-frequency range (10–200 Hz).

19.10.4 Countermeasures against vibrations and ground-borne noise

The impacts of ground-borne noise and vibrations are limited when

- They are restricted to the initial vibrations
- The range of frequencies within which they are generated is far from the natural frequency of the system which they affect (the building frame, the track bed system, etc.). Solutions that guarantee frequencies at least $\sqrt{2}$ times the natural frequency of the system are considered effective. Indicatively, it is stated that the floating slab systems (mass spring systems) act as a barrier against vibrations, with a frequency that is $\sqrt{2}$ times higher than the natural frequency.

In any case, the measures taken at the source, and specifically at the railway track, are the most economical and effective.

One of the basic principles applied is the increase of the mass of the track which is supported elastically. In this way, the natural frequency is reduced, and thus the efficiency of the system is increased, in terms of insulation against vibrations.

The technical solutions that can be applied for the reduction of vibrations and ground-borne noise are (Wettschureck, 1995, 2002; Eadie et al., 2002; Liu, 2005; Vogiatzis, 2006; Eitzenberger, 2008; Kuo et al., 2008) given below.

In all track bed cases

- Elimination of the geometrical track defects
- Elimination, mainly, of the wave corrugations of rails (by grinding the rails)

- Use of high-resilience pads and fastenings, for the absorption of vibrations in the first place
- Elimination of wheel flattening
- Use of resilient wheels (Figure 19.14)
- The increase of the width of the tunnel ring
- The increase of weight of the tunnel
- The increase of the mass of the track superstructure supporting system (mass of concrete slab or sleepers)
- Wheel slide protection systems
- Lubrication of the inner side of the rails in curved sections of the horizontal alignment that have a small radius (reduction of squeal noise)
- The increase of the strength of the rail and the use of heavier rails
- Vehicle bogies with low vertical stiffness springs on the level of the primary suspension vehicles with light unsprung and semi-sprung masses
- Placement of vibration-damping materials in the rail web (by welding) (web dampers). With this technique, a reduction of vibrations (of VAL) up to 2–5 dB can be attained
- Continuous welded rails and movable point frogs on track crossings
- Placement of switches and crossings, where possible, away from land use that is sensitive to vibrations
- Reduction of the train passage speed

In cases of ballasted track

- Maintaining the ballast thickness, in any case, to more than 30 cm
- Interference of layers of resilient or elastomer materials under the ballast layers (sub-ballast mats)
- Placement of elastic pads between the sleepers and the ballast. A reduction of VAL by 15 dB at 125 Hz was recorded
- Foamed ballast

It has to do with stabilising the ballast, by using special polyurethane foam, toward the increase of its durability:

- High vibration-damping sleepers
- High-width sleepers

In the case of slab track
We distinguish virtually two techniques:

- *The solution of the floating slab*: This technique was described in Section 5.3.2.
- Other solutions that attach a more resilient vertical behaviour to the superstructure and its support on the concrete slab. Such solutions (Nelson, 1996; Eadie et al., 2002; Cox, 2003; Vanhonacker and Leuven, 2005) are the following:
  - Resiliently supported twin-block sleepers
  - Resilient direct fixing fastenings of rails
  - Resilient pre-load APT-ST direct fixing system
  - Super-resilient pre-load APT-BF direct fixing system of rails (Vanhonacker and Leuven, 2005)

19.11 IMPACTS ON LAND USE

The railway systems, like all other transport systems, ensure access conditions among various land uses and serve the mobility needs arising from the interaction of the activities taking place within them. This way the railway systems influence

- The location of land uses in relation to proximity access to the transport network
- The composition of land uses in the sense of relationships and interactions that are created through the links provided by the transport network
- Land value, which is formed by the combined effect of accessibility and composition of land use

The type and characteristics of the effect on the location and the composition of land uses and land values depend on the operational characteristics of the transport system. Thus, at the level of urban development, the areas around the main stations of the urban railway transport modes are developed featuring high density and mixed land use, combining housing, services, trade and recreation, while the regions that are mainly served by the private car showcase trends toward suburbanisation, urban sprawl and zoning of land uses, such as industrial zones and major shopping centres (Newman and Kenworthy, 1999). On the contrary, the location of households or businesses in the areas around train stations is considered to be a competitive advantage leading to increased land values (Debrezion et al., 2006).

19.12 COMPARATIVE ASSESSMENT OF THE IMPACTS OF VARIOUS MEANS OF TRANSPORT TO THE NATURAL ENVIRONMENT

19.12.1 Methodology approach

In order for a rational comparison to be made, between railway systems and other means of transport, it should relate to transport systems of similar functionality and performance, that is, systems that can be considered as competitive, and provide roughly the same level of service to their users.

In this context, the following comparisons take place in this paragraph:

- For very long-distance trips (500–1,500 km), the aeroplane and the high-speed railway are compared with each other.
- For urban movements, the metro, the tram, the private cars and the urban bus are compared with each other.
- For high-speed transport, the aeroplane, the high-speed train and the magnetic levitation train are compared with each other.
- For the modes involved in freight transport, the freight train and the camion are compared with each other.

19.12.2 Long distances: Comparison between the aeroplane and the high-speed train

On the basis of the study conducted in 2010 by JR Central in the Tokyo–Osaka line, with a length of 515 km, the high-speed train (Shinkansen Series N700 'Nozomi') consumes about 1/8 of the amount of energy consumed per passenger/seat by the aeroplane (B777-200) and emits about 1/12 of the CO_2 emissions of the aeroplane per passenger/seat (Garcia, 2010a).

Table 19.13 Emission of pollutants in the Frankfurt–Hannover line

Data per passenger	*Aeroplane*	*High-speed train*
Energy consumption (equivalent in petrol litres)	32.8	11.1
Carbon dioxide (kg) Greenhouse gases Global warming	77.1	19.2
Particulate particles (gr)	2.1	1.0
Sulphur dioxide (gr)	43.4	19.5
Nitrogen oxides (gr)	268.3	17.2
Hydrocarbons, excluding methane (gr)	20.8	1.1

Source: Adapted from Kettner, J. 2008, *Future Challenges of Transport and Environment: The Role of Railways to Reduce Climate Gas Emission*, Deutsche Bahn AG.

In a study conducted by the IFEU (Institute for Energy and Environmental Research), a comparison was performed in the Frankfurt–Hannover connection, between the aeroplane and the high-speed train, in terms of emission of pollutants. Although in Germany more than 50% of the required energy for train operation is generated by fossil fuels, the high-speed train, because of its high efficiency, emits much less pollutants, as is demonstrated in Table 19.13 (Kettner, 2008).

From Table 19.13 it is concluded that, for the specific connection, the high-speed train emits 4 times less CO_2 and 16 times less NO_x per passenger.

Table 19.14 displays the relationship of the most significant ambient air pollutants for the high-speed train and the aeroplane. These results refer to the implementation of eight connections, which were investigated and taken from the computer software available online at www.ecopassenger.org. For this calculation, the following assumptions were drawn (see Section 15.3.1):

- Regarding the train, it was assumed to be full of passengers.
- Regarding the aeroplane, the gas emissions of the modes that are used for the transfer from and to the airport were also taken into account.

As it is also demonstrated in Figure 19.16, the most annoying noise is caused by the aeroplane, followed by that of the road, and, finally, the noise from the railway. More specifically, from the persons exposed to a noise level of 55 dB, which is critical, according to the World Health Organization (WHO), 30% considered aeroplane noise as annoying, 20% considered the noise produced by road means of transport as annoying, while only 10% were annoyed by the noise of the railway (Den Boer and Schroten, 2007).

Table 19.14 Summary of very long-distance trips

Comparison parameter	*Comparison quantity*	*Relationship between high-speed train/aeroplane*
Carbon dioxide	g/pkm	1/7
Nitrogen oxides	g/pkm	1/9
Hydrocarbons	g/pkm	1/18
Energy consumption	Equivalent fuel in L/pkm	1/4

Parameters of comparison and ratio.

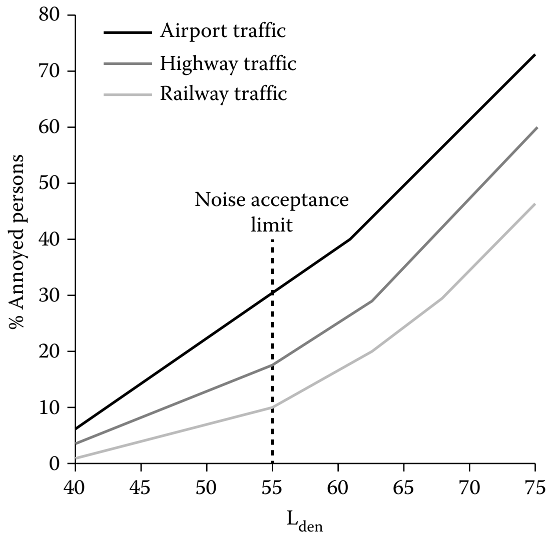

Figure 19.16 Percentage of annoyed persons from the traffic noise caused by various means of transport. (Adapted from Den Boer, L.C. and Schroten, A. 2007, Traffic noise reduction in Europe, *CE Delft Report*, 2007.)

The data held by the EU are consistent with the aforementioned, since it is concluded that the highest percentage of disturbance is derived by the noise of the aeroplane, at 25%, whereas the lowest percentage is assigned to the railway, at 10%.

The noise caused by high-speed trains is a short-term annoyance, which affects the environment locally, when the train crosses a specific area. In contrast, the noise caused by aeroplanes lasts longer, since high-noise levels are generated in the areas around airports, both during landing and takeoff, and when the aeroplane flies at a low altitude.

19.12.3 Urban transport: Comparing the metro, tram, urban bus and private car

The bar chart of Figure 19.17 graphically displays the distance (in km) travelled by various means of urban transport, based on the consumption of 1 kgr of fuel equivalent.

As demonstrated in Figure 19.17, the suburban railway and metro cover almost the same distance, over twice (2.5 times) that of the private car and 1.2 times that of the urban bus.

In the United States, a light metro vehicle with 55 passengers is estimated to require 0.19 kW h/pkm, an urban bus with 45 passengers consumes 0.2 kW h/pkm, and a private car with a driver and three passengers requires 0.33 kW h/pkm. In contrast, a private car carrying only his driver consumes about 1.34 kW h/pkm.

Table 19.15 displays the parameters compared for the four means of urban transport, and the relationship among them, for each of these parameters.

Table 19.16 shows the noise level of various vehicle types, and the noise pollution of various train types, for a specific distance between the track centre and the receptor.

It can be concluded from Table 19.16 that the bus causes the highest noise pollution, and the metro and the tram the lowest. Nevertheless, since the noise levels of the various urban means of transport are similar, in order for the railway to become more competitive, efforts should be made to reduce noise pollution and the vibrations caused by them.

19.12.4 High-speed transport modes: Comparisons of the aeroplane, the high-speed train and the magnetic levitation train

With regard to the comparison of high-speed means of transport, the comparison results are displayed in Table 19.17.

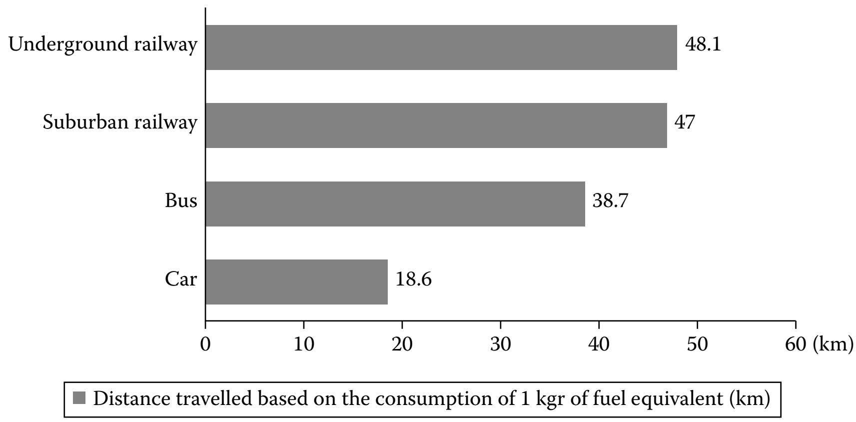

Figure 19.17 Energy efficiency of urban means of transport.

Table 19.15 Comparative table for urban movements

Comparison parameter	*Comparison quantity*	*Relationship among means of transport*	
Nitrogen dioxide (NO_2)	gr/100 pkm	Metro/urban bus 1/3	Metro/private car 1/4
Carbon monoxide (CO)	gr/100 pkm	Metro/urban bus 1/189	Metro/private car 1/311
Energy consumption	kW h/pkm	Light metro/urban bus 1/1.1	Light metro/private car 1/7
Energy consumption	Distance travelled, based on the consumption of 1 kgr of fuel equivalent	Metro/urban bus 1.2/1	Metro/private car 2.5/1

Parameters of comparison and ratio.

Table 19.16 Noise levels of various vehicle types, and noise pollution of various train types, for a specific distance between the noise receptor and the track centre

		Distance between noise receptor–railway track axis		
Vehicle type	*Noise level (dB)*	*7.5 m*	*15 m*	*25 m*
Passenger cars with a maximum capacity of five seats	77	–		
Passenger transport vehicles (buses) with more than nine seats and weighing more than 3.5 t	80–83	–		
Buses with weight up to 3.5 t	78–79	–		
Short-length train, metro, and tram	70–80	79	75	72

Table 19.17 Comparative table for high-speed movements

Comparison parameter	*Comparison quantity*	*Relationship among means of transport*		
Carbon dioxide		Magnetic levitation train/aeroplane	Magnetic levitation train/ high-speed train	High-speed train/ aeroplane
		1/5.5	1/1.5	1/3.7
Energy consumption	BTU/passenger-mile	Magnetic levitation train/aeroplane	Magnetic levitation train/ high-speed train	High-speed train/ aeroplane
		1/2.25	1/1.39	1/1.62
Noise pollution	dB (A)	Magnetic levitation train/aeroplane	Magnetic levitation train/ high-speed train	High-speed train/ aeroplane
		1/1.2	1/1.125	1/1.06

Parameters of comparison and ratio.

Table 19.18 Comparative table for freight movements

Comparison parameters	*Comparison quantity*	*Relationship of freight train/road truck*
Carbon dioxide	100 tkm	1/4.7
Nitrogen oxides	100 tkm	1/19.3
Energy consumption	kW h/100 tkm	1/3.3

Parameters of comparison and ratio.

19.12.5 Freight transport: Comparison of freight trains and road trucks

Table 19.18 displays the comparison parameters, as well as the comparison results.

Table 19.18 shows that the freight train has 4.7 times less carbon dioxide emissions than the road truck. Furthermore, it consumes 3.3 times less energy.

In freight transport, CO_2 emissions for the railway are 24 gr/tkm, while for the camion, the respective value is 88 gr/tkm. Compared with the camion, CO_2 emissions of the railway are almost 4 times less.

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Chapter 20

Cutting-edge technologies in railways

20.1 DEFINITION AND CLASSIFICATION OF CUTTING-EDGE TECHNOLOGIES

The term cutting-edge technology implies a technology which

- Is fairly recent (it has been applied or investigated in the last 10–15 years)
- Is innovative
- Improves to a great extent, through its application, the performance of a system
- Constitutes a significant field of research and application

Table 20.1 pinpoints railway technologies that can be characterised as 'cutting-edge technologies'. For each particular technology, the following are presented:

- Its name
- A short description
- The development status (whether it is in the stage of research or in use and the date of first application)
- The railway systems to which is applied
- The aims of its development and the fields that it improves
- The technologies which it replaces or which it is an alternative to

In the following sections, selected cutting-edge technologies of Table 20.1 are described and analysed in detail.

20.2 SMART WINDOWS

The operation of 'smart' windows is based on the use of innovative synthetic glasses that permit automatic adjustment of their optical and thermal properties, depending on external conditions. These windows absorb solar radiation and re-emit it inside the vehicle, ensuring natural lighting and heating, and allowing, at the same time, the user to control the amount of light and heat that permeates the glass (Figure 20.1).

Smart windows make use of various technologies:

- Thermotropic
- Photochromic or photochromatic
- SPD (suspended particle devices)
- Electrochromic
- Reflective hydrides

Table 20.1 Cutting-edge technologies in railway transport systems

Name of technology	*Short description*	*Development status*	*Year of first application*	*Railway transport systems where it applies*	*Alternative technologies*
APS system	Contact-based ground power supply of trains	In use	2003	Tram	Overhead catenary system
TramWave system	Contact-based ground power supply of trains	Pilot application (line in service)	November 2014	Tram	Overhead catenary system
PRIMOVE system	Induction-based ground power supply of trains	Test stage completion	–	Tram	Overhead catenary system
Energy storage devices	Battery or supercapacitor-based power supply of trains	In use	2007	Urban systems	Overhead catenary system
Smart windows	Automatic adaptation of the optical and thermal properties of window glass panes to the external conditions	In use	2014	All systems	Conventional glass
Carbon fibres	Vehicle parts made of composite light materials	In use	1990s	All systems	Steel, aluminium, carbon steel, and stainless steel
Glass fibres	Vehicle parts made of composite light materials	In test stage	–	All systems	Conventional materials
LRC (laser railhead cleaner) system	Laser for cleaning fallen leaves on the rails	In test stage	–	All systems	High-pressure water Sandite-sand held in gel Plasma torch Leaf deflectors
Hydrogen trains	All types of rail vehicles which use hydrogen as an energy source	Completion of the construction of various prototypes	2015 (streetcars in Aruba and Dubai, tram in Qingdao, China)	All systems	Environmentally friendlier energy sources
Energy-efficient train control / energy-efficient driving strategies	Decision support systems to optimise energy consumption of rail systems	In use – further development	–	All systems	
Communications-Based Train Control (CBTC)	Movement authorities via wireless communication systems	In use in urban railway systems	2003('90s)	Urban railway systems	Traditional signaling systems based on track circuits and line side signaling

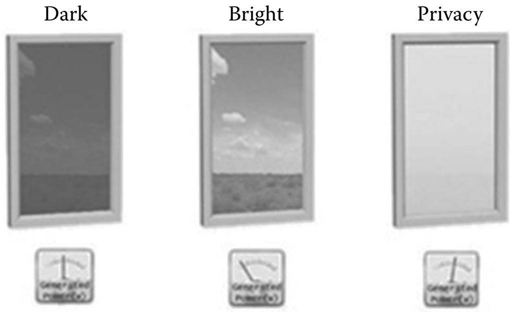

Figure 20.1 Operation of smart windows. (Adapted from Online image, available online: http://www.greenpacks.org/2010/03/15/smart-energy-glasses-generate-energy-stay-smart/ (accessed 7 April 2015; From Peer+B.V., 2015.))

Thermotropic and photochromic technologies cannot be controlled manually. Thermotropic materials become darker by reacting directly with the sunlight and, as a result, their use in winter is not recommended, when it is required to heat the interior of the vehicles. Photochromic windows react with the heat (become darker) and, as a result, during the summer months, passengers have limited view.

The last three technologies are the most interesting ones and are currently evolving.

SPD technology allows the window glass to convert its surface from transparent to non-transparent, and also to any intermediate condition, automatically or manually with a switch (Bonsor, 2010; Katanbafnasab and Abu-Hijleh, 2013).

Windows with the above properties were used for the first time in a test train route in Japan, on the 20 April 2014 (Research Frontiers, no date; Railway Gazette, 2014a).

Electrochromic windows use special materials, which, along with an electrode system, may change colour when electric current permeates them. In essence, electrical power causes a chemical reaction that changes the properties of these materials. In specific electrochromic materials, this change relates to many colours. In electrochromic windows, however, the changes are between coloured, which reflects the light of a colour, and transparent, which reflects no light at all.

In an electrochromic window, electrical power is only required to implement the first change in transparency. The maintenance of a specific colour shade does not require constant voltage. Minimum voltage is required to implement and also reverse the initial change. This is also the major element that makes it very efficient in terms of energy. Depending on the construction parameters, full colouration and discolouration may need from 2 s to 10 min.

Reflective hydrides may be described as electrochromic materials; however, they behave in a different way. Instead of absorbing light, they reflect it. Thin films made of a nickel–magnesium alloy are able to change a transparent condition to a non-transparent one, and vice versa. This change can be supplied with high-voltage electric current (electrochromic technology) or the diffusion of hydrogen and oxygen gases (aerochrome technology). These materials are considered to be more efficient in terms of energy than other electrochromic materials (Bonsor, 2010).

Below are presented the advantages (with +) and disadvantages (with –) of smart windows, compared with windows using conventional technology (Bonsor, 2010):

- (+) Control, either automatic or manual, of the intensity of natural lighting. Adjustment by the staff and also by the passengers
- (+) Lower weight (by 30%) and size
- (+) Better soundproofing of the interior of the vehicle

- (+) Lower installation and maintenance cost. Reduction of the operating cost (cost of heating, lighting, air conditioning, and abolish the use of sunscreens)
- (+) Higher thermal insulation capacity
- (+) Reduction of reflections
- (+) Quick change from transparent to non-transparent status
- (+) Protection against ultraviolet radiation, and therefore, protection of the materials of the vehicle's interior space against wear and discolouration
- (+) Resistance against a high range of temperatures
- (+) High lifespan
- (–) Significant initial investment
- (–) Use of electric charge

20.3 CARBON AND GLASS FIBRES

Composite materials are increasingly applied to the railway, in a wide range of fields. The excellent relationship between resistance – specific weight of composite materials, such as carbon fibres and glass fibres, their resistance against fatigue caused due to vibrations and repetitive charges, and resistance against water, corrosion and chemical effects, make these materials ideal both for the railway infrastructure and the rolling stock.

Carbon fibres consist of very thin fibres with a diameter between 0.005 and 0.010 mm. These fibres are made mostly of carbon atoms, which are joined into miniscule crystals placed parallel to the long axis of the fibre. The final element is in the form of fabric (Figure 20.2)

One of the main advantages of the use of composite materials is the reduction in the tare of vehicles by up to 65% compared with the case of using steel, and up to 45% in the case of using aluminium.

The use of glass fibres is still in test stage, while the limited use of carbon fibres so far is estimated to expand to a great extent in the near future. It is stated, indicatively, that carbon fibres are used more and more in the front cabs, rear ends, fairings, floors and under frame coverings of the carbody shell of various urban railway vehicles (in France, Italy, Great Britain, Austria, and also in the metro of London, Lille and Hong Kong).

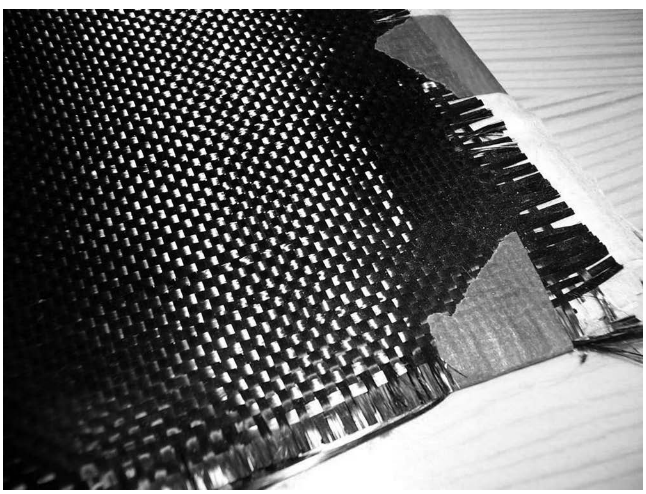

Figure 20.2 Fabric made of carbon fibres. (Adapted from Hadhuey at German Wikipedia. 2005, Online image available at: https://commons.wikimedia.org/wiki/File:Kohlenstofffasermatte.jpg (accessed 8 August 2015).)

20.4 LASER RAILHEAD CLEANER SYSTEM

An issue encountered in many railway networks is the presence of 'leaves on the track'. Fallen leaves on the track superstructure cause wheel sliding that may lead to long delays, and even cancelled services. According to the British company Network Rail, leaves on tracks caused 4.5 million hours of passenger train delays in 2013 (Gray, 2014).

Wheels compress leaves, causing a layer of black slippery and hard substance like Teflon on the rolling surface of the railhead. The reduction of adhesion affects the rolling, braking and signalling systems.

The solutions used nowadays toward this issue are either high-pressure water jetting or jetting a mixture of sand and gel called sandite, by ejectors placed on the trains. Both methods have disadvantages, since trains have to be supplied on a constant basis with the necessary materials, while severe corrosion can be caused on rails.

The first attempts to use a laser in order to clean railway lines are dated in 1999 in Great Britain, without, however, delivering the desired results.

In November 2014, the Dutch started retesting a system that burns leaves with a laser. Lasers are placed right in front of the wheels and target low, so that, as the train passes, they burn the accumulated mass of the leaves. Moreover, they dry the rails, preventing the accumulation of new leaves.

The LRC (laser railhead cleaner) system (Figure 20.3) has been installed on a test basis on train DM-90 of the Dutch Railways. The wavelength used is absorbed only by the leaves and other organic materials that may be found on tracks.

The laser is deactivated temporarily in case it loses its target. So far, researchers have improved the system, so that it can clean up to 20 mm of substances on the rail, on both sides of the track, and at speeds that may reach 80 km/h (Daily Mail, 2010; Brown, 2014; Structon Rail, no date).

20.5 CATENARY-FREE POWER SUPPLY OF TRAMWAY SYSTEMS

The power supply systems of a tramway are divided into three major categories: overhead power supply, ground power supply and energy storage (Table 20.2).

These systems can be used on their own for the operation of a tramway system (as autonomous systems) or can be combined with each other (mixed power supply or hybrid systems).

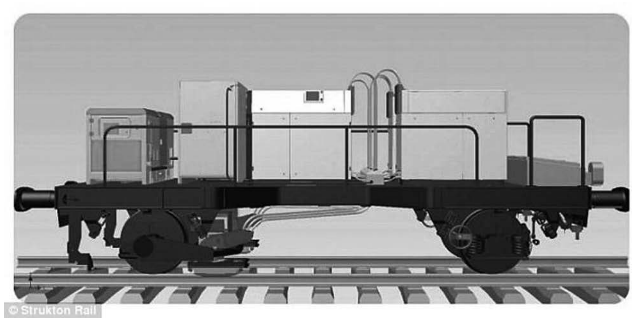

Figure 20.3 The LRC (laser railhead cleaner) system. (From Structon Rail, 2015.)

Table 20.2 Classification of power supply systems used in tramway networks

Major categories of power supply systems used in tramway networks	*Subcategories*
Overhead supply	• Trolleybus type • Catenary type
Ground supply	• Conventional systems of third and fourth rail • APS • TramWave • PRIMOVE
Energy storage systems	• Supercapacitors • Batteries • Flywheels

In Table 20.3 are presented all the potential combinations, and for each one, the systems that are currently either in operation or under construction. The potential combinations are expressed with the symbol √.

The above data as well as the data recorded and analysed in the following relate to the year 2014. The raw data were obtained from various available sources and cross-checked. Afterwards they were further manipulated for the needs of this chapter.

In the following are described in detail the systems of ground power supply and energy storage which are considered as cutting-edge technologies in the railway sector, as they meet

Table 20.3 Potential combinations of tramway systems power supply – systems in operation and under construction – recording based on the power supply system

	Overhead power supply (catenaries)	*APS*	*TramWave*	*PRIMOVE*	*Supercapacitors*	*Batteries*
Overhead power supply (catenaries)	√ The overwhelming majority of tramway networks	√ Bordeaux, Angers, Reims, Tours, Orleans	√ Beijing	√	√ Zaragoza, Shenyang, Seville	√ Nice, Konya
APS	√ Bordeaux, Angers, Reims, Tours, Orleans	√ Dubai			√	Inability to recover energy from braking
TramWave	√ Beijing		√ Zhuhai		√	√
PRIMOVE (conductive transfer)	√			√	√	√
Supercapacitors	√ Zaragoza, Shenyang, Seville	√	√	√	√ Zhuzhou	√ Doha
Batteries	√ Nice, Konya	Inability to recover energy from braking	√	√	√ Doha	Nanjing (in a percentage of 90% of line length)

the requirements of Section 20.1. All these technologies are relatively new and are in various development stages. Flywheels are in a very primary stage. However, flywheels show a series of disadvantages that prevent their extensive use in railway applications, while they have a relatively high weight (SIEMENS, 2012; CCM, 2014).

20.5.1 Ground power supply systems

20.5.1.1 The APS system

The APS system (Alimentation Par le Sol – electrical power supply of trains on ground level) was developed by Alstom, for the supply of electricity through a 'third rail' to trams. The third rail is literally a carrying conductor integrated into the ground and placed between the two main rails of the track.

The system operates with electrified segments at a voltage of 750 V, and length of 8 m, separated by neutral segments of 3 m length (Figure 20.4).

The required energy for the movement of the tram is ensured in the following way: each train (articulated tram) has two electricity collectors (collector 'shoes'), in the middle car, close to which is placed an antenna, which sends radio signals to 'power boxes' that supply the conductor with electric current only when the tram passes just over it, resulting in the elimination of the risk of electrocution to pedestrians, cyclists, animals, etc. (Figure 20.5). Power boxes are placed every 22 m. At the same time, the batteries placed on the roof of the vehicles are charged in the track sections where there is overhead power supply, and use the stored energy when power supply from the ground is out of order due to 'power box' failure, and also in the areas of intersection with roads. The energy that is stored in the batteries permits autonomous movement of up to 50 m (Novales, 2010; Cooper Bussmann, 2012).

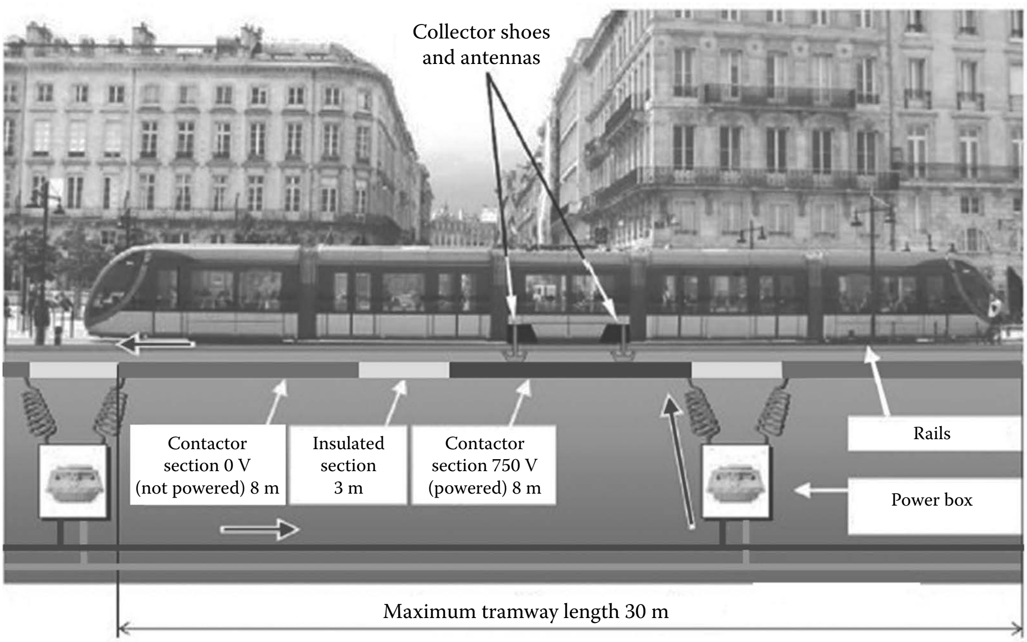

Figure 20.4 Operating principle of the APS system. (Adapted from Novales, M. 2010, Overhead wires free light rail systems, *90th TRB Annual Meeting 2011*, Washington, DC, 23–27 January, available online: http://assets.conferencespot.org/fileserver/file/32564/filename/16668k.pdf) (accessed 11 April 2015; From Alstom Transportation, 2015.)

Figure 20.5 The APS system is totally safe for pedestrians, cyclists and road vehicles. (Adapted from Finn, B. 2007, *Tram system in Bordeaux report on the tram system and underground power supply (aps) for Dublin city business association*, December 2007, available online: http://www.dcba.ie/wp-content/uploads/2012/04/Bordeaux-Report-07.12.07.pdf (accessed 11 April 2015).)

The minimum required length of line, in order to apply the APS power supply system, is 30 m. Conduction segments of 8 m length are activated and deactivated thanks to an isolation switch, which is located inside the embedded power box.

The specific technical characteristics of the APS system are summarised below:

- The tram collects power from the pairs of contact shoes placed under the central car of the tramway. The shoes are 3.2 m away from each other, which means that a tram cannot be decoupled in the neutral segment of the line of 3 m.
- The process of lowering the pantograph for the movement of the train by the APS technique is performed at specific stops during boarding/alighting of passengers.
- At road intersections insulations are placed on rails. The total length of the insulated track sections at crossings is 5 m or more; therefore, when a tram is decoupled from the catenaries at a crossing, it should be supplied by the batteries in order to proceed to the next electrified track segment.
- Electric current ceases to flow through the conductor after the passage of the tram. Therefore, if more than two consecutive electrified track segments are identified, the automatic power switch cuts off electrification in both segments, and its electrification cannot be recovered unless manually (activation normally takes about 20 min after isolation) (Mott MacDonald Group, 2008).

The APS system has already been applied in practice, and the issues that occurred initially have been overcome (Finn, 2007). Specifically, it was applied for the first time in 2003 to the tram network of the city of Bordeaux, France, in 13.6 km out of the 44 km of the network, and in particular within the historic centre of the city (Figure 20.6).

Nowadays, the APS system is used in six tram networks worldwide (Table 20.4). The tram of Dubai, the construction of which was completed in November 2014, is the first tram in the world to have, throughout its length, the APS (autonomous system) ground power

Figure 20.6 The tram of Bordeaux, where the APS system is used within a part of the network. (Adapted from Pline, 2008, Online image available at: https://commons.wikimedia.org/wiki/File:XDSC_7576-tramway-Bordeaux-ligne-B-place-des-Quinconces.jpg (accessed 8 August 2015).)

supply system. In the other five systems, the APS system operates only in parts of the network, in conjunction with the conventional overhead catenary system (mixed power supply system) (see Table 20.3).

The advantages of the APS system against other conventional solutions with overhead wires are

- Its superiority with respect to aesthetics, as it relieves from visual annoyance
- The ability to perform all activities that were prevented before due to the presence of overhead wires
- The performance of trains with regard to speed and dynamic comfort of passengers, which are relevant to those with conventional power supply systems

Table 20.4 Tramway systems that use APS technology (2014 data)

City (country)	*Line length (km)*	*Cost/km million (€)*	*Operation starting date*	*Percentage of line length equipped with overhead catenary system (%)*	*Percentage of line length equipped with APS (%)*
Tours (France)	14.8	27.8	2013	86.5	13.5
Orleans (France)	11.8	30.8	2012	78.8	21.2
Angers (France)	12	32.9	2011	87.5	12.5
Reims (France)	11	34.1	2011	81.8	18.2
Bordeaux (France)	44	35.7	2008	72.7	27.3
Dubai (UAE)	10	66.8	2014	0	100.0

Source: Adapted from *Railway Gazette*. no date, available online: http://www.railwaygazette.com/ (accessed 12 July 2014); Wikipedia. 2015a, Fenbahn, available online: http://de.wikipedia.org/wiki/Stra%C3%9Fenbahn (accessed 15 July 2014); Wikipedia. 2015c, Tramway, available online: http://fr.wikipedia.org/wiki/Tramway (accessed 22 July 2014); Light Rail Now. no date, available online: http://www.lightrailnow.org/ (accessed 20 August 2014).

Its disadvantages against conventional solutions with overhead wires are

- The potential reduction of the performance in case of accumulation of rainwater.
  The requirement for immediate removal of water in case of flooding, results in a significant increase of the maintenance and operating cost of the network, whereas the requirement for placement of protective covers and insulating materials on the current-carrying conductor, for protection against moisture, increases the infrastructure implementation cost.
- The requirement for a high level of tightness in the isolation equipment of a segment, which does not permit their placement in a soil with a high aquifer level, for example, on the banks of a river.
  In terms of their protection, their latest version is based on the EN 60529 standard and they can be kept submerged in 1 m of water for 15 days.
- When the tram is driven by the APS system, for safety reasons, braking with energy recovery is not possible (non-compatible with the regenerative-braking technology). Alstom investigates the improvement of energy efficiency through the on-vehicle installation of supercapacitors (SYSTRA, 2012).
- This technology has been implemented so far by Alstom in the Citadis 302 and 402 type vehicles.
- Problems in the presence of snow, frost and sand.
- A design that should take into account the mechanical stress caused by road traffic. In the case of application to the city of Bordeaux, the railway line shares infrastructure with road traffic, and as a result, it should endure charging for years. In other French cities, the APS system is mostly installed in a protected corridor, with the exception of course of intersections, where the corridor is necessarily common.
- Finally, the higher total implementation cost of a tramway system which uses the APS system, compared with the cost of a tramway system runs exclusively on conventional power supply
  Specifically, the use of APS technology, instead of the overhead catenary system, increases the following three components of the total project cost as follows):
  - The installation cost of the traction system
    The cost of the power supply equipment placed on the line in the case of using APS technology is seven times higher per metre of length than the cost of installation of overhead catenary system
  - The cost of rolling stock
    By convention, an increase should be taken into account in the region of 10%–15% of the cost of conventional rolling stock, when alternative power supply is used from the ground. The installation in retrospect of APS equipment on conventional power supply vehicles requires modifications (installation of a collection device, installation of batteries on the roof, installation of antennas under the frame of the bogies, and modification of the traction circuit). The layout of the substations is identical for both APS and conventional power supply.
  - The cost of works with regard to the superstructure, the substructure and the restoration of the road infrastructure.

According to Alstom Transportation the increase of the overall implementation cost (rolling stock + power supply + track + civil engineering) in comparison to the conventional solution (overhead catenary system) is estimated to

- 3%–4% in case of a double track line of 12 km length with 2 km APS (18.5%)
- 10% in case of a double track line of 11 km length with almost all with APS (100%) (Tram of Dubai)

20.5.1.2 The TramWave system

The TramWave system has many similarities to the APS system. It has been developed by Ansaldo. It supplies a voltage of 750 V through a continuous tubular conductor encased in the ground, which is placed between the two main rails of the track (Figure 20.7).

The power supply line is implemented by a series of current-carrying conductor segments that are insulated from each other, each of which has a length of 3 or 5 m. The segments are supplied with current only when the train passes over them. The presence of trams is detected by gravity sensors and other electrostatic means.

The energy collector is placed under the vehicles, whereas the power collector is placed in the ground within the power supply conductor. A ferromagnetic zone in the power supply conductor on the ground allows electrical energy to be transmitted to the train. The track segment with electric charge is very small – just 1.5 m – in order to maximise safety. As soon as the train passes from the specific segment, electricity supply is cut off.

The TramWave can be adjusted in many types of vehicles, including those with rubber-tyred wheels, and operates simultaneously with the conventional overhead catenary system. When the overhead system is in operation, a safety system sets the power collector placed in the ground to 'off' mode.

The TramWave can be combined with energy storage systems (ESS), and the supply of electricity can be used in order to recharge it. In this way, the vehicle can be disconnected from the power supply cable, and continue autonomously with the energy stored on the vehicle.

Ansaldo asserts that this system has lower maintenance requirements than conventional overhead catenary systems, because any type of malfunction can affect only one tram, which is detected automatically by the diagnostic system, and can be replaced within just 30 min. Furthermore, the immobilisation of a tram does not prevent other trams from circulating on the lines.

Figure 20.7 The conductor of the TramWave system. (Adapted from ANSALDO STS. no date, *TramWave ground-levelpowersupplysystem(nooverheadlines)*,Italy,availableonline:http://www.ansaldo-sts.com/sites/ansaldosts.message-asp.com/files/imce/tramwave_eng.pdf (accessed 11 April 2015).)

The TramWave completed successfully the test stage in Naples (successful tests on a test line of 400 m length, pilot operation within the city of Naples, in a partly separated corridor of 600 m length), while its application in practice is anticipated in two cities. Specifically

- On 7 November 2014, its operation started on a test basis in a segment of the new under-construction line of 8.7 km length, in the city of Zhuhai, China, with the total line expected to be completed and open for traffic within 2015. The power supply of the trains is performed exclusively by the TramWave system.
- In 2015, operation of the Xijiao line is anticipated in the west part of the city of Beijing. Out of the total 9.4 km of the route, catenary power supply will be applied to 5 km, as well as to the depot, while the other segments will be supplied on ground level by the TramWave system.

In Table 20.5 are presented the tramway systems which use or which are going to use TramWave technology.

The differences between the TramWave and the APS system are the following:

- TramWave is a technology that has been applied only to one line (test operation) in practice, and hence there are not any credible data in terms of the cost and reliability of the system.
- It can be adjusted to many vehicle types, including those with rubber-tyred wheels.
- The return current is transferred through the power supply conductor. In this way, the TramWave eliminates the effects of leakage currents, avoiding the requirement for insulation of the track infrastructure.
- There is the ability of energy recovery during braking (compatible with regenerative-braking technology).

The equipment of the TramWave system has been installed only on Sirio-type vehicles of Ansaldo. Upgrading the conventionally supplied trams for the operation of the TramWave system requires modifications on the vehicle.

20.5.1.3 The PRIMOVE system

This system has been developed by Bombardier. Like the other two systems, it has power supply equipment placed in parallel to the two main rails of the track. However, in this system, ground power supply is performed inductively and not by contact. The power supply is used, either for the operation of electric motors, or charging of storage devices that may be placed on the vehicle.

Table 20.5 Tramway systems which use or which are going to use TramWave technology

City (country)	*Line length (km)*	*Operation starting date*	*Percentage of length operating with overhead catenary system (%)*	*Percentage of length operating with the TramWave system (%)*
Zhuhai (China)	8.7	7/11/2014 (test operation in line segment) 5/2015 (total line)	0	100
Beijing–Xijiao line (China)	9.4	2015	53.2	46.8

In the PRIMOVE system, the tramway infrastructure has, in every 8 m, a separate segment of loops of inductive coil of 8 m length, which transfers power energy inductively to the train (Figure 20.8). These inductive coils (primary inductive loop) are fully embedded in the pavement. Their covering is performed with various materials, such as asphalt or concrete. Below the underground induction coil there is a magnetic shield which prevents electromagnetic interference. Inverters are placed along the line and are supplied with a direct current of 750 V DC, and convert it to alternating current which can be used in the PRIMOVE system. When a high-frequency alternating current flows through a segment of induction coil loops, it causes an electromagnetic field. The segment is activated by the detection and control cable of the segment of induction coil loops (vehicle detection and PRIMOVE segment control [VDSC] cable), which detects the presence of a vehicle over it.

The vehicles have induction receivers under the floor (secondary inductive loop), which capture the inductive power that is transferred by the primary inductive loop and a compensation condenser (the PRIMOVE power receiver system), and which converts the electromagnetic field to alternating current. On-vehicle inverters convert alternating current to direct current used for its power supply. They also have, on the roof, energy storage devices, and specifically, the MITRAC energy saver, which uses a double layer of capacitors. The capacitors store the released energy during braking and also from PRIMOVE's infrastructure both during train movement and at stops for future use. It can also be combined with Li-ion batteries, leading to the optimisation of the system's energy efficiency and also the reduction of the required infrastructure. The detection of cable segments is performed by the detection antenna of the vehicle and control of induction coil loop segment (VDSC antenna) on the vehicle, which coordinates their activation/deactivation.

Just like in the TramWave or the APS, the cable segments are supplied with power only when the train is over them, which makes the PRIMOVE system safe for pedestrians and road vehicles.

The system is able to supply 100–500 kW of power. It can be used for the operation of trams of 30–42 m length, on a ground slope of up to 6% and for speeds of up to 90 km/h, with 270 kW of power (Brecher and Arthur, 2014).

Since the power supply is performed without contact, the system operation is not affected by snow, ice or sand.

The PRIMOVE system was installed on a test basis in Augsburg, Germany, in 2011, in a segment of 0.8 km length. The pilot application was completed successfully in June 2012.

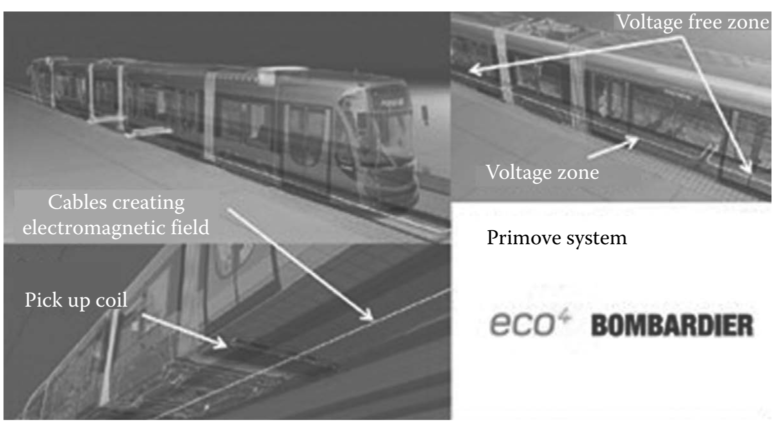

Figure 20.8 The Primove system. (From Bombardier Transportation, 2015.)

The differences of the PRIMOVE system, compared with TramWave and APS systems, are the following:

- Reliability even under adverse weather conditions (snow and ice), since power supply is performed inductively.
- Compatibility with all types of pavements.
- There is also, like in the TramWave system, the ability of energy recovery during braking (compatible with regenerative-braking technology).

It is a technology that has not been applied in practice yet, and hence there are not any credible data about its cost and reliability.

20.5.2 Power supply systems with energy storage devices

The main disadvantage of overhead and ground power supply systems is the fact that they require the installation of fixed infrastructure, with subsequent impact on the implementation and maintenance cost (Novales, 2010). An alternative solution is the use of ESS on the vehicle. The initial goal of this technology was not to avoid the use of overhead wires, but to improve the energy efficiency of light railway systems.

Indeed, a way to reduce energy consumption of a tram is through regenerative braking, which has been expanded widely to railway systems. Regenerative braking, in order to be effective, required simultaneous presence of other vehicles in close distance from the braking vehicle for the absorption of surplus electrical energy, which would otherwise return to the overhead catenary system, especially when power supply was performed with direct current. Overheating of the overhead wires caused by the total energy generated by the motors during the braking operation used to be a frequent event.

To prevent such an event, trams started to get equipped with integrated ESS, for recovery and using braking energy when necessary, such as in case of acceleration. Very shortly, technicians realised that this technology could be used in order to avoid the use of overhead wires in network segments.

The process of charging can be accomplished with three different methods:

- By the overhead catenary during conventional operation
- By the regenerative-braking process
- By installation of charging infrastructure in stations

ESS can be divided into three types of applications:

- *Mobile storage applications:* Mobile storage applications consist of onboard ESS usually located on the vehicle roof. Every system works independently and the recovered energy is directly sent to the storage system placed on the vehicle. When the vehicle accelerates, energy is used in priority from the ESS to propel the vehicle.
- *Stationary storage applications*: Stationary storage applications consist of one or several ESS placed along the tracks. These devices recover the exceeding energy when no other vehicle is receptive meanwhile.
- *Stationary back to the network applications*: Stationary back to the grid applications whose main difference with the previous applications is that they do not store the recovered energy but send it to the main electrical network for other consumers such as lighting, escalators, administrative and technical buildings or potentially sold back to the energy provider.

Charging of the storing devices can be achieved by the following methods:

- *Regenerative-braking process*
- *Charging from power supply cables*
  In this case, the batteries placed on the vehicle are charged by the overhead catenary system during the stage of conventional power supply, in order to store the necessary amount of energy for autonomous movement in the areas where the overhead catenary system has been abolished to achieve an improved aesthetic result. There is also the possibility of charging from ground power supply, as it applies in the case of PRIMOVE.
- *Charging from power supply cables installed at stops*
  The charging process is implemented at stops through an overhead catenary system which has been installed only in the area of stops. In this case there is the requirement for rapid charging of storage devices (which is an advantage of supercapacitors over batteries). Charging can also be attained from the infrastructure which is installed inside the rails (CSR Zhuzhou).

20.5.2.1 Supercapacitor charging/ESS (supercapacitors or ultracapacitors)

Supercapacitors comprise an improved and more competitive version of capacitors used in applications of means of transport. Supercapacitors are identical energy storage means to electrochemical cells, that is, batteries, with energy however being stored as electric charge, instead of chemical reactors, as it happens in batteries. High-capacity batteries are able to store large quantities of energy; however, their charging takes long. Not only are supercapacitors charged faster than batteries, but they also last longer, because they do not suffer from natural wear of charging and discharging, which exhausts the batteries. The charging of supercapacitors is implemented within a few seconds (20 s), which allows them to be charged at stops, during the boarding and alighting process.

Supercapacitors allow braking energy recovery, resulting in energy saving percentages of up to 30%.

In Table 20.6 are listed the major differences between batteries and supercapacitors.

The biggest disadvantage of supercapacitors, compared with lithium-ion batteries, is their much smaller energy density. Lithium-ion batteries may store up to 20 times the energy of supercapacitors for a given weight and size.

Supercapacitors are placed on tram roofs. This solution implies the increase of vehicle weight and cost, but it is offset by the saving of energy (Gonzalez-Gil et al., 2013).

New materials, such as grapheme, could ensure high energy density in capacitors, and fully replace lithium-ion batteries.

The purchase cost of supercapacitors is very high; however, it is expected to be reduced by the dissemination of technology.

The benefit enjoyed from the use of supercapacitors, as well as of batteries, urges manufacturers to adopt hybrid systems that use both technologies (see Section 20.5.2.2).

In Table 20.7, the tramway systems are presented which use or which are going to use supercapacitors.

The company of CSR Zhuzhou from China presented in mid-2014, in its statement, one of the first trams to move exclusively through supercapacitor technology, and not use any aerial conductors in contact, for its power supply (Figure 20.9). The trains carry supercapacitors on their roofs, which are sufficiently charged when trains make their scheduled stops. CSR adds that the vehicles draw sufficient power during a stop of 10–30 s, which

Table 20.6 Major differences between batteries and supercapacitors for their use in tramway systems without overhead wires

Features	*Batteries*	*Supercapacitors*
Lifespan	20,000 charging cycles	100,000 charging cycles
Braking energy recovery	Very low	Yes
Indicative cost	20% of vehicle cost + replacement cost in the end of lifespan	20% of vehicle cost + replacement cost in the end of lifespan
Charging time	High	Low
Required time between two consecutive charges	High	Low

Source: Adapted from Global Mass Rapid Transport. 2014, *Catenary-free trams:Technology and recent developments*, April 1, 2014.

Table 20.7 Tramway systems which use or which are going to use supercapacitors exclusively or in combination with other systems

System	*Company*	*Application cases*	*Power supply*
ACR	CAF	Zaragoza (2013)	Overhead catenary system + supercapacitors + batteries
ACR	CAF	Seville (2011)	Overhead catenary system + supercapacitors + batteries
	CSR Zhuzhou	Zhuzhou (China) (12/2014)	Supercapacitors (they are charged at stops from ground power supply)
Sitras-HES	Siemens	Doha (Qatar)(2015)	Supercapacitors + batteries (auxiliary operations) (they are charged at stops from overhead power supply)
	Changchun Railway Vehicles Company	Shenyang (China) (2013)	Overhead power supply + supercapacitors
Sitras-HES	Siemens	Almada-Seixal (2008)	Overhead power supply + supercapacitors + batteries

allows them to operate independently for up to 4 km. The supercapacitors provide power to the traction system for the acceleration of the tram, and they also draw energy from the 'braking energy recovery' system, which can recover up to 85% of the braking energy to be reused. The vehicles are charged automatically, through a device that is placed between the rails, which is activated as vehicles pass over the track (Railway Gazette, 2014b).

Figure 20.9 Prototype tram by the company of CSR with supercapacitors. (Adapted from Nissangenis, 2014, onlineimageavailableat:https://commons.wikimedia.org/wiki/File:Guangzhou_Haizhu_District_CSR-Zhuzhou_Tram_For_No.05.jpg)

20.5.2.2 Charging/ESS with batteries

The use of batteries as ESS is a tested and mature technology, which provides a relatively good ratio of weight/power, and it is low cost. One of the major disadvantages of the system is that the battery is affected by the ambient temperature. At low temperatures, a loss in capacity is noted, while at high temperatures, bending of the plate can be caused, which leads to loss of voltage, or even to an event of fire in extreme cases.

By using fully charged batteries, a distance of 30 km can be covered in autonomy. Hybrid technology allows batteries to be recharged by the catenaries, when the train is supplied by the traditional overhead electrification.

Batteries are vulnerable to fast charging cycles and deep discharges, which lead to the reduction of their expected lifespan and total performance. Another disadvantage is that most of the batteries require regular maintenance and inspection to keep their performance on a high level.

Nevertheless, the rapid development of nickel–metal hydride (Ni-MH) batteries in the last years offered to several tram manufacturers an alternative solution both to overhead and ground power supply systems. In November 2007, Nice became the first city in France to use batteries for running tram vehicles in its network. Each vehicle is equipped on its roof with Ni-MH batteries charged by a conventional overhead catenary system, which allow CITADIS vehicles to move autonomously inside the two historic squares of the city. The batteries allow the vehicle to operate at speeds of up to 30 km/h, however, at a lower

Table 20.8 Tramway systems which use or which are going to use batteries in combination with other systems

System	*Company*	*Battery type*	*Autonomy*	*Application cases*	*Power supply*
Swimo OCS-free vehicles	Kawasaki	Ni-MH	10 km	Test stage (Sapporo)	Batteries + overhead catenary system
CITADIS	Alstom	Ni-MH	1 km	Nice (2007)	Batteries + overhead catenary system
LFX-300 ameriTRAM	Kinkisharyo	Li-ion	8 km	Demonstration stage (Dallas, 2011)	Batteries + overhead catenary system
CATFREE drive	Skoda Transportation	Nano-lithium–titanium batteries	8 km	Konya (2015)	Batteries + overhead catenary system
PRIMOVE batteries	Bombardier	Li-ion	90% of the length of the line (throughout the line excluding the area of stops)	Nanjing–Hexi line (2014)	Batteries (90% of the line length) + overhead catenary system (10% of the line length (at the area of the stops)

Source: Adapted from Akiyama, S., Tsutsumi, K. and Matsuki S. 2008, The development of low floor battery-driven LRV 'SWIMO', Kawasaki Heavy Industries, Ltd., Kobe, Japan, May 2008; Vuchic, V. R. 2007, *Urban Transit Systems and Technology*, John Wiley & Sons, ISBN: 978-0-471-75823-5, p. 624; Railway Technology. no date, b, *Shenyang Tramway, China,* available online: http://www.railway-technology.com/projects/shenyang-tramway/ (accessed 21 August 2014); Railly News. 2013, Bombardier partner CSR Puzhen to supply catenary-free trams to Nanjing, available online: http://www.raillynews.com/2013/bombardier-partner-csr-puzhen-to-supply-catenary-free-trams-to-nanjing/ (accessed 17 August 2014); Marotte B. 2011, *Building a New Canada – Bombardier's transit vision pairs efficiency with wireless technology*, available online: http://www.theglobeandmail.com/news/national/time-to-lead/bombardiers-transit-vision-pairs-efficiency-with-wireless-technology/article4181172/ (accessed 17 August 2014).

acceleration rate than in segments with overhead catenary system. Their operating life is 5 years, allowing trams run in autonomy for up to 1 km. The change from overhead supply to batteries is activated by the train driver, while the pantograph lowers when the vehicle moves without overhead supply (Railway Technology, no date, a).

Manufacturers have invested in the potential of lithium-ion batteries which offer higher energy storage density than Ni-MH cells, and which have achieved significant development in the sector of rolling stock manufacturing.

In Table 20.8 are presented various examples of tramway systems using batteries.

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Transport Engineering

"This book is thoroughly researched and clearly presented with lots of helpful and well-illustrated examples. …a very useful resource for railway engineers and anyone needing information on railway systems from around the world."

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This textbook presents a comprehensive overview of railway passenger and freight transport systems, from design through to construction and operation. It covers the range of railway passenger systems, from conventional and high speed inter-urban systems through to suburban, regional and urban ones. Moreover, it thoroughly covers freight railway systems transporting conventional loads, heavy loads and dangerous goods. For each system it provides a definition, a brief overview of its evolution and examples of good practice, the main design, construction and operational characteristics, the preconditions for its selection and the steps required to check the feasibility of its implementation.

The book also provides a general overview of issues related to safety, interface with the environment, cutting-edge technologies and, finally, the techniques that govern the stability and guidance of railway vehicles on track.

It suits students, and also those in the industry — engineers, consultants, manufacturers, transport company executives — who need some breadth of knowledge to guide them over the course of their careers.

Christos Pyrgidis is a professor of railway engineering in the civil engineering department at Aristotle University of Thessaloniki, Greece. He has participated in many research railway projects and studies and has worked as a consultant for several railway organizations. From 2004 to 2007 he was the Greek representative to the Administrative Board of the European Railway Agency (ERA). Professor Pyrgidis is a member of the Scientific Committee of SHIFT2RAIL Joint Undertaking (S2R JU).

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