DEPARTMENT

OF

CIVIL

ENGINNERING

CE6604 - Railways, Airport and Harbour Engineering

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The Definition of Railway Rolling Stock

It is always useful at the outset of consideration of a subject to pause for a moment and to ponder the definitions, attributes range and scope of the matter.

Rolling stock used on railways in the earliest days evolved from carriages and wagons which ran on highways to carry both people and bulk materials.

As early as the sixteenth century wooden wheeled carts were used in mines and quarries running on longitudinal timber rails. With the progressive evolution of the skills and crafts of the wheelwright, metalworker and the ironmaker, wheels improved through various phases from simple rough

turned wooden spools through spoked and rimmed construction to fully cast and turned metal wheels.

Similarly, body construction and springing, particularly for passenger carrying vehicles, relied very heavily on the experience gained in the construction of stagecoaches in the seventeenth and eighteenth centuries. At the end of the eighteenth century, horse drawn trams running on metal rails began to appear in a number of European cities. These horse drawn tramways were literally to pave the way for development of railways when steam power began to be developed

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early in the 1800s. One has only to look at illustrations of early passenger coaches to see how closely they resemble
the road vehicles of the previous century.
As railway experience was gained, the design of rolling stock also evolved. Springing, body structure, wheels and axles all are subject to varying loads and stresses, when comparing slower speeds on rough roads to much faster speeds on railways, with a comparatively smoother ride.
Railway rolling stock generally runs on hard wheels on hard rails. The wheels are not only supported by the rails but are guided by them. The only exception to this is for a small number of metros where rubber tyres have been introduced. In this case the supporting function of the rail may be separated from the guiding function.
In all cases railway rolling stock will transmit vertical, horizontal and longitudinal forces to the track and its supports. Most railways have adopted twin rails and flanged wheels. Forces are transmitted to the rail
structure either by direct bearing on the rail top from the wheel tyre, or by bearing laterally through the flange, or by longitudinal friction. Potential ‗overturning' forces, caused by centrifugal force on curves, coupled with wind forces on exposed locations are resisted by vertical dead weight and
super-elevation or ‗cant' on curves.

The Range of Railway Rolling Stock

Today there is a very wide range of rolling stock used throughout the world
on different railways. This range includes the following basic types:
• Locomotives
• Freight wagons
• Passenger coaches
• Multiple units (with motive power in-built)
• Metro cars (usually multiple units)
• Light rail/Trams (usually articulated units)
• Rail mounted machines (cranes, tampers etc.)

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• Inspection and maintenance trolleys

The Objectives in Station Planning

In planning any station the following objectives need to be kept very much
in mind:

• Attractiveness in appearance.
• Free movement of passengers.
• Safe evacuation in emergency.
• Access for the disabled.
• Access for emergency services.
• Safe accumulation and dispersal of crowds.
• Reliable operation of train service.
• Resilience to failure.
• Cost-effective investment.

Planning for Normal Operation

The degree to which the business is prepared to invest in providing space purely for the added comfort of passengers must be decided by each railway system based on its own market position and objectives. The starting point for any station planning is the demand forecast. This must be accompanied by a detailed knowledge of the likely train frequency from each platform and the time staff would need to take action when problems arise. Given working assumptions, it is then possible to determine
how many people are likely to have accumulated within a particular area before control measures can be instituted.

The operator must determine his own relative values for key variables which combine to determine the minimum size and capacity for any element of a station.

These will include:

• time needed to become aware of a problem.

• staff reaction and decision time.
• action implementation time.
• accumulation rate for passengers.
• maximum density for safety.

The frequency and destination pattern of the train service is also a key factor in the sizing of station infrastructure. Assuming, for instance, that the total staff reaction time is effectively five minutes and that the normal peak service is at five minute intervals, capacity at the platform must allow
for at least twice the normal numbers expected in the peak.

Capacity Requirements

It is recommended that the following limits should be applied to station areas for demand levels under normal peak conditions:
Platforms, ticket halls and concourses — 0.8 sqm per person
Passageways
• one way — 50 persons per minute/m width
• two way— 40 persons per minute/m width
Fixed Stairways
• one way — 35 persons per minute/m width
• two way— 28 persons per minute/m width

To allow for ‗peaks within a peak' it is wise to use the calculated peak fifteen-minute flow figure, which can be derived from the one-hour figure by multiplying by 0.3.

Similarly the peak five-minute flow figure can be derived by multiplying the fifteen-minute figure by 0.4. This five-minute figure should be used when testing the layout ‗tight spots' to ensure that dangerous situations do not occur during the short lived period when crowding exceeds desirable levels
at a restricted localised point.

The capacity of entrances and exits to street level should follow the guidelines above. From subsurface ticket halls/concourse areas there should be at least two exits to the street each of which must be able to take the full peak level demand albeit under crowded conditions.

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Locations which are fed by exits from stations need to be examined to ensure that no bottle-necks exist immediately outside station buildings.

This is particularly important where stations exit into Local Authority subways, shopping malls or where sporting events are likely to produce ‗tidal wave' crowding.

The Evolution of Steam Motive Power

As has been mentioned previously, the harnessing of steam power in the late eighteenth and early nineteenth centuries was the springboard for the development of railways throughout the world. The concept of running hard rimmed flanged wheels on narrowmetal rails had been tried out in the mines

and quarries and found to be both workable and advantageous.

The main limitation to the effectiveness of using plate-ways, rail-ways or tram-ways was the adequate provision of haulage power or what became known as ‗motive power'.

Walking pace motive power was first provided by men and horses and later in some places by stationary engines driving winches for cable hauled cars. As the design of wheels, axles and bearings steadily improved, towards the end of the eighteenth century, heavier loads could be moved and rail

borne movable steam ‗locomotives' became a possibility.

The first steam hauled train was operated by Richard Trevithick's steam locomotive in South Wales in 1804. While this locomotive seems to have worked quite well on a mine tramway, the cast iron plates that formed the track proved to be inadequate for the heavier loads and impacts.

Hard on its heels, William Hedley's ‗Puffing Billy' built in 1813, ran on a tramway near Newcastle-on-Tyne giving successful service for over forty years.

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The first use of steam for a passenger train was George Stephenson's _Locomotion' on the Stockton and Darlington Railway in 1825. There is a wall plaque at the original railway station at Stockton which reads:

The first public railway to use steam motive power exclusively and to run a regular passenger service was the Liverpool and Manchester Railway which commenced operations in 1829. This railway was perhaps the first to have the essential elements of a modern railway. All trains were locomotive hauled, running to a timetable, operated by company staff and only stopping at stations manned by its own staff. The railway linked the two cities and was only 38 miles long, taking about two hours six minutes to do the journey. This average speed of 18 mph seems extremely slow to us but when compared to walking, running, or going by narrowboat or stagecoach, was a substantial improvement.

What is even more amazing is that fourteen short years later Daniel Gooch, locomotive superintendent of the Great Western Railway, drove Prince Albert home from Bristol to London in about the same time, a distance of about 118 miles! The average city to city speed on that journey of 57 mph is still remarkable and could not be achieved today by driving from Bristol to London, even with the fastest car, without breaking the speed limits!

During the rest of the nineteenth century railways continued to develop and spread to all parts of the civilised world. With this development both steam locomotives and all types of rolling stock grew in size and complexity.

Steam power dominated traction on most of the worlds railways in the first hundred years or so. Indeed, until the 1880's, steam was the only form of motive power that was considered viable for railways. Even

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the so called _atmospheric‘ railways still relied on stationary steam engines to provide
their power.
In the very earliest days, even at the time of George Stephenson‘s _Rocket‘, boilers were fitted with multiple tubes, water space round a fire box and a fire which was drawn by the exhaust steam blasted up the chimney. Most locomotives had two cylinders linked to the large driving wheels by external connecting rods.
Cylinders were normally inclined at an angle to the horizontal and drove only one pair of wheels. Eventually cylinders were placed horizontally in a forward location and the driving power was linked to all the _driving wheels‘ by various cranks and connecting rods.
There was also a great deal of activity in the design and evolution of valve gear, slides, pumps and pistons which all added to both the efficiency and the complexity of steam locomotives. Steam traction is simple in essence and some complexity led to more difficulties and problems than were solved.
The invention of _super-heating‘ of steam in the late nineteenth century led to adoption of this feature in later steam locomotives, giving rise to higher efficiency but also a need for better maintenance, particularly of boilers and tubes.
Early underground railways adopted steam power for hauling train because at that time there did not appear to be any practical alternative.
The first underground railway in the world was opened by the Metropolitan Railway Company in 1863 between Paddington and Farringdon, London. By that time many hundreds of miles of main line
railway had been built around the world and over thirty years experience had been gained in the design, manufacture and operation of steam locomotives.
This original section of the new line, together with its later extensions (now the Circle Line), was constructed using the _cut-and-cover‘ method. As the construction was only at a shallow depth, openings

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were left wherever possible in an attempt to ensure steam, smoke and fumes were adequately
ventilated.
The original intention was to use conventional steam locomotives on this line burning no fuel on the underground sections but relying on them‚_head of steam‛ and heating up only at the end of the comparatively short underground section.
When the line was opened it was found that conventional locomotives caused distress to passengers and staff due to the discharge of carbonic oxide gases. Some relief of the problem was found in construction of condensing engines but clearly some other form of motive power would be desirable
underground. The London commuter had to suffer the inconvenience of steam locomotives in confined spaces for another three decades or so before
a satisfactory alternative was found.

The Advent of Electric Traction
The possibility of electric traction was first demonstrated by a Scotsman
called Davidson in 1834 but it was not until the Berlin Exhibition of 1879
that the idea was developed far enough to show that it could be a practical
challenger to steam.
The obvious advantages of electric traction over steam for underground
railways attracted the attention of many engineers and operators around
the world in the last decade of the nineteenth century.
The first _Tube‛ line to be built in London was the City and South London Railway between King William Street and Stockwell in 1890 using

electric traction. This was followed within ten years by the construction
of the Central London Railway from Shepherds Bush to Bank, also using
electric traction. Other tube lines followed rapidly, all of which
were incorporated into today's London Underground.
Most of these early tube lines followed the main line practice of a single
locomotive pulling non-powered carriages or cars. The City & South London locomotives were small four wheeled vehicles whereas the Central
London Locomotives were a much larger ‗camel back' design with four
driving axles mounted in two bogeys.
During the first decade of the twentieth century all of the London tube lines departed from the principle of single locomotive hauling to using a
number of motorcars along the length of the train. This has considerable
advantage for rapid transit trains, not the least of which is to distribute both traction and braking along the full length of the train. This has the
effect of improving both acceleration and braking, which is important on
lines where there are frequent stops.
For the same reasons many main line railways have now come away from
the use of locomotives for suburban and stopping services and have adopted
multiple units with motors distributed along the length of the train.

Development of Electric Traction

The suburban and underground railways that were built or electrified in

the early part of the twentieth century adopted a medium voltage direct
current supply system which involved fairly costly fixed equipment but kept
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the locomotives relatively simple and cheap. A large number of transformer
‗sub-stations' were involved with comparatively heavy conductor rails set
at track level. Technology was very similar to the early electric tramways
which were also powered with direct current.
In the UK, London Underground and a large part of the Southern
Region of British Railways adopted DC electric traction many years before
the rest of BR converted from steam power to diesel power or seriously
considered large scale electrification.
Overhead supply of high voltage alternating current was pioneered
largely in Switzerland after the First World War and by the 1930's became
the normal system of electrification on the Continent.
High voltage AC electrification was not introduced to British Railways
until after the Second World War since when it has become the preferred
system for surface railways. High speed AC electric locomotives have a high
power/weight ratio as they carry no heavy fuel.

Diesel Traction

This alternative form of motive power was invented by a certain Doctor

Diesel of Berlin in about 1893. There are specific technical problems associated with applying diesel power to railways. These mainly relate to the fact that the engine must be turning even when the locomotive is stationary, unlike the steam engine which has latent power provided the head of steam is up. In road vehicles this can be overcome by the familiar mechanical device of introducing a clutch and gearbox. This works well for vehicles of moderate horsepower but is unsatisfactory for more powerful engines. Because of this drawback, the diesel engine was relatively late in coming to the railway scene. It was not until the 1930's and later that the diesel began to be taken seriously and only in the 1950's that diesel and electric traction finally ousted steam in most parts of the developed world. Two main methods of coupling the diesel engines to the driving wheels were evolved and still remain today. The first involved hydraulic drive which had modest success. Most of this type of locomotive originated from Germany and many are still running today.

Without doubt however, the standard diesel locomotive today throughout the world is the diesel-electric. One could describe this as an electric locomotive with its own on- board diesel generator power station. The solution of the drive problem is complicated and therefore expensive.

As a very rough indication of this, the first cost of a diesel locomotive

is about three times the cost of a steam locomotive of similar power. However
the real savings come to light when considering the ‗whole-life' costs involved in running and maintaining steam versus diesel.
In particular, steam requires many man-hours each day before and after
working to get up the fire and rake out the ashes etc. The diesel locomotive
has immediate push button power and has a much lower requirement for
‗down-time' for regular maintenance.

Evolution of Wheel Layout

The earliest steam locomotives had two or three axles, one or more of which carried the driving wheels. Richard Trevithick's locomotive had an
ingenious arrangement which connected the two driving axles to the driving
pistons by means of a series of large cog wheels.
In many cases the inclined cylinders drove one pair of large driving wheels directly and these were sometimes linked to other wheels with ‗connecting'
rods. As locomotives grew in size, weight and power additional wheels were introduced largely to carry the extra weight of water and coal
which was needed for the ever increasing journey length. Locomotive designers
needed to get as much weight onto driving wheels as reasonably possible
to avoid wheel slipping or spinning, a characteristic of steam engines. Heavy
individual axle loads however were most undesirable from the point of view
of supporting brides and structures. As in all engineering design, this has

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always meant that some compromise needs to be made between operational desirability and practical structural considerations.

The introduction of electric and diesel multiple units has allowed the use of many more driving wheels along the length of a train, thus reducing the adhesion, acceleration and braking problem.

Changes in Locomotive Maintenance Practices

Steam traction involved the procurement of many extra locomotives because of the large amount of cleaning, lubrication, descaling and minor repairs that were necessary.

This resulted in all engines spending a considerable proportion of their life in the shops or sheds instead of out on the railway pulling trains. Additionally there was a lot of time spent in firing and other preparation before each day‘s working.

It was these considerations, amongst others, that led in the UK and many other countries to the demise of steam and the gradual introduction of diesel and electrical power since the SecondWorldWar. Diesel and electrical power has also enabled designers to dispense with large driving wheels and to introduce power driven bogies.

Also in recent years both locomotives and multiple units have been designed with motive power packages and self contained units that can be removed or replaced for maintenance. This has the effect of reducing yet further the amount of time that trains or locomotives have to be out of service.

‘Carriages’ to the Modern Carbodies

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Third class passengers were often carried in simple wagons very little different from ‗cattle trucks'. Sometimes as many as three or four stagecoach
bodies were mounted on one truck with the seats ‗facing or back'
towards the engine. The doors on these early coaches were on each side, one
per coach compartment, with no connection between the compartments.
The carriages on suburban stopping trains on BR retained an element of the same layout with individual unconnected compartments and single
‗slam' side doors for well over a hundred years. As railway journey times
and distances increased, this quickly evolved for ‗express' trains, with the
introduction of a side corridor, to the basic carriage layout which remained
normal for main line railways in the UK to the 1960's.
Early American railways, however, adopted the open coach with passengers
sitting each side of an open corridor, boarding and alighting from the train through doors at the ends. Many of the world's railways have now
adopted the open plan for both main line and suburban services with doors
at carriage ends and walk-through connection down the full length of the
train.
Most metros and light rail systems have open plan layouts in the cars. In this case however boarding times are critical and doors only at car ends
would be too restrictive. In this case there is a very fine balance between
the number of doors and the number of seats provided.

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The levels and curvature of platforms also has an effect on the design of cars and this varies considerably around the world. Increasing consideration is being given to the need to accommodate disabled passengers, especially those in wheelchairs.

Carbody Structures

Since the earliest days of railways, carbody structures have evolved and become considerably stronger, lighter and more economic. As mentioned previously, the earliest carriages were largely of wooden construction. These proved to have a very low crash resistance when accidents occurred with a high rate of injury and loss of life.

As early as 1840, in the UK the Railway Inspectorate was set up to inspect newly constructed railways and to certify fitness for public travel.

Various accidents investigated by the Inspectorate over the years have led to progressively higher standards being set for the design of rolling stock.

The first stage was to introduce a wrought iron and later steel underframe which fully supported the wooden superstructure. This system lasted well and was still being used in new stock up to the 1950's. The main drawback was that all the strength was in the chassis which performed well in collisions but body work splintered, still causing much loss of life and physical injury.

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The next stage was the use of a steel underframe with steel or aluminium
framing to the superstructure. This performed much better in crashes but
the whole design was getting very heavy and expensive.
Modern cars and coaches are designed on the _Monocoque' principle.
In this case the whole structure is designed as a single monolithic unit, spanning between the main bogie supports. The structure then takes all
the bending, shear and torsion stresses as an entity.
The final form is usually a composite of aluminium extrusions and welded stainless steel with a _stressed skin'. All loads and stresses are distributed
between the various components. The resulting design is considerably lighter than the previous design and is much akin to aircraft structural design. The lighter design coupled with higher stresses and repeated loading
means that fatigue considerations become increasingly important.
Summarising, these developments of carbody design over the last almost
two centuries are characterised by:

- Lower mass
- Higher stiffness
- Higher strength

These rolling stock characteristics lead to:

- Lower energy consumption
- Greater crashworthiness
- Higher passenger comfort
- Higher passenger/carbody mass ratio

Main Line Train Performance Issues

When considering the engineering of a railway from the rolling stock point

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of view, train performance demands and issues need to be fully considered.
These vary according to location and whether or not it is Main line, Metro or Light Rail.
The performance issues on Main Line railways for consideration are as follows:

- Is the traffic mainly one type (e.g. high speed express passenger) or mixed speed and type?
- What will be the impact on the long distance passenger carrying capacity of the railway of slow freight and stopping trains?
- What capacity will the signalling allow? (This will depend largely on such factors as the length of the signalling sections and whether there is uni-directional or bi-directional signalling.)
- Are there many passing loops or _slow line' platforms at stations of secondary importance, to allow expresses to pass?
- What acceleration, braking characteristics and tractive effort is required to ensure that trains can work to desirable timetables?
- What are the maximum gradients on the line? (These will effect the previous consideration greatly.)
- How many speed restrictions are likely and what recovery will be required of time lost?

3.12 Train Performance Issues on Metros and Light Rail

Generally metros and light rail systems only carry passengers and there is

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not therefore the added complication of _mixed‘ traffic. Similarly, in most
cases, there is no segregation of express and stopping trains to complicate
matters. The main concern is to provide trains which will closely match
demand at various times of the traffic day in the most economic manner.
The main issues therefore are:
• What capacity is required at various times?
• What are the achievable acceleration and deceleration rates?
• What _dwell time‘ is required at stations?
• What top speed is necessary?
• How can energy be conserved in normal running conditions e.g. by coasting?
• How much scope is required for recovering lost time in the peak due to delay?
• For light railways only, what additional factors need to be taken into consideration for any lengths of track which are incorporated into the highway and where _shared running‘ takes place?
Once all these considerations have been fully investigated, decisions can
be made on the type and number of different units of rolling stock that is
required to run the railway. To this there must be added extra stock to allow
for the fact that there will always be some vehicles on programmed repair
and maintenance as well as others out of service for unplanned reasons or
mishaps. With modern rolling stock and proper maintenance procedures,

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this extra proportion should be able to be kept to not more than a quarter,
depending on the size of fleet.
It can often be shown that a small amount of extra capacity, both in tractive effort and braking, can play dividends in the long run and allow
overall economies in energy.
For relatively short distances between stops, the timetable for trains running normally should allow for a pattern of motoring up to maximum
allowable speed and then coasting for a period before braking for the next
station. This pattern is less demanding on energy than flat-out speed and
maximum braking which can howeverbe resorted to if lost time needs to
be regained due to delay.

Freight Rolling Stock

Early railways were characterised by ‗goods' trains of a very mixed variety.
In the days of steam it was commonplace to see long trains of mixed wagons
carrying coal, stone, timber, slate and many other basic materials needed
both in the large cities and in the smaller towns. The operation of such trains was often slow and labour intensive, involving marshalling yards and
painstaking ‗shunting' and off-loading.
Economic considerations have caused such operations now to be a thing
of the past. However, railways are still an excellent way of moving freight

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especially in large countries where distances are much greater. Even so approaching two hundred million tons of freight a year is still transported on the railways in the UK.

Freight wagons in recent years have tended to become specialised to the material they are handling. This is certainly the case for the transport of bulk cement, china clay, crushed stone, coal, oil, steel, fly-ash and some manufactured items like cars.

In recent years also specialised fixed formation trains have been used in the UK, known as freightliners, which run on regular routes from ports and various factories carrying standard containers.

Some of the larger dedicated bulk carrying vehicles that run in the UK have twin wheeled bogies and a total ‗all-up' laden weight of up to

100 tonnes. This has the effect of producing a train which imposes 25 tonnes axle loadings down the full length of the train. This is very punishing to the supporting track and structures and must be taken into account by all engineers engaged both in vehicle and bridge design.

Specialised Engineering Rolling Stock

Railways were originally very labour intensive. This applied particularly to the civil engineering activities involved in both laying and maintaining the permanent way and its supporting earthworks.

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Early etchings of railway building activities show that a great deal was achieved by sheer weight of manpower assisted only by hand tools, wheelbarrows and the trusty horse. Contractors often laid temporary track on which they used their own small steam locomotives hauling simple wagons.

On these tracks some used simple steam powered mobile cranes but that was about the limit of mechanical plant available.

On railways today engineers have designed many items of plant, both stationary and mobile, which reduce considerably the manual tasks associated with keeping the track up to a good standard.

All specialised engineering rolling stock has to comply with all the safety, signalling and operational requirements on the railway. Some is self propelled and can be treated as a train operating in its own right. Other plant is hauled to site as part of a train and only operated under its own power within the confines of a complete possession of the railway. Specialised vehicles included the following:

- Ballast tamping machines
- Ballast cleaners
- Ballast hopper wagons
- Stone blowers
- Mobile rail cranes
- Long welded rail cars
- Cleaning trains
- Inspection cars/trolleys
- Snow and leaf clearing vehicles
- Concreting trains
- Drain/sump cleaners

• Battery cars/Ballast locomotives
• Tunnel cleaners
• Platelayers' trolleys
• Personnel carriers
• Track recording cars
• Rail grinders
• Special flat cars/bolster wagons for track.

Manufacturing Methods

Originally railway rolling stock was manufactured using simple engineering skills with most components being _bespoke'.Manufacture was labour intensive which was relatively cheap. In more recent years multiple engineering skills have become involved with more specialisation, complex design and use of standard components. Skilled labour has become progressively more expensive in real terms. Additionally there have been a number of major changes in manufacturing technology.

These changes include the following:

• Riveting has been replaced by welding.
• There is an increase in the use of aluminium and stainless steel.
• Plastics have been introduced.
• There is a greater use of jigs and fixtures.
• Computerised manufacture and production control.
• Introduction of quality assurance.

The Origin and Development of Railway Track

Before the beginning of the eighteenth century wheeled transport was generally hauled by horse and ran on surfaces which at the best was reinforced

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by a broken stone foundation and at the worst was simply a mud track. It was found at a very early stage of the development of land transport, that most road surfaces and foundations were very quickly damaged by heavy wagons on rigid wheels.

The first railway tracks were laid down in the eighteenth century for horse drawn trains of wagons in collieries and quarries. These _hauling ways‘ initially had a surface of stone slabs or timber baulks which proved unsatisfactory as loads grew heavier. As the Industrial Revolution progressed the idea was developed further by adding wrought iron plates to reduce wear on the wooden baulks. This evolved further first to cast iron plates and later to edge rails, enabling for the first time the use of flanged iron wheels.

By the time locomotives came on the scene in the early nineteenth century, wrought iron rails had developed further and became strong enough to support these heavy engines without assistance from longitudinal timbers.

In 1825 the Stockton and Darlington Railway was constructed adopting track of wrought iron rails resting in cast iron chairs supported on stone blocks set in the ground at three feet intervals. The rails were of _T‘ section 15 feet long and weighed about 28 lbs per yard.

As experience was gained and new technology evolved, rails steadily increased in size, both in length and cross section, and were made in steel

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rather than iron. Early railways evolved the _bullhead‘ or dumbbell section
of rail which was standard throughout the UK up to the SecondWorldWar.
This rail was manufactured in increasing lengths and heavier sections and
by the early 1900‘s had been generally standardised to 60 foot lengths and
about 95 lbs per yard weight. Most railways today use flat-bottomed rail.
The individual stone block sleepers were early found to be unwieldy and
unsatisfactory from several points of view, largely relating to weight and the
lack of tying of rails at a fixed gauge. These blocks were quickly replaced by
timber cross sleepers which proved to be much more economic and satisfactory.
Cross sleepers, or _ties‘ as they are known in some countries, have been
generally adopted worldwide and are now often manufactured in concrete
or steel although timber is still used extensively. At a very early stage the
need for good preservation of softwood left in wet ballast became very obvious.
By the 1880‘s several railway companies had set up their own plants
to impregnate sleepers with creosote under pressure.

Basic Components of Track

Today most railways have rolling stock with hard steel flanged wheels running
on two rails set at or about 1432mm standard gauge, supported
in some way to spread loads to the ground below.

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‗Sub-grade‘ is the term used for the natural soil stratum, or embankment soil, after trimming off organic topsoil and made ground, upon which the track bed is constructed.

The ‗Trackbed‘ comprises the ballast and any sub- ballast layers and is there to support the track, to drain water from the bottom of the sleepers and to distribute the imposed track load to such a degree that the sub-grade can resist the imposed bearing pressure adequately.

5.3 Track Ballast

Early railway engineers did not at first realise the important engineering function carried out by the ballast, as outlined above. Because of this all manner of material was used beneath the sleepers which today would be considered completely unsuitable. This included materials which would be cheaply and easily available locally such as ashes, chalk and clay.

Experience soon showed that good quality ballast, made of well graded gravel, crushed gravel, limestone or igneous rock was necessary if adequate foundation and good drainage is to be achieved for a reasonable period.

Additionally it was found that, even with good quality crushed material, the presence of a high proportion of ‗fines‘ in track ballast can quickly result in silting up and softening of adjacent and supporting materials, thus causing track settlement and drainage long term problems.

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Today the required depth of good quality ballast beneath sleepers varies depending upon the maximum speed of trains, the maximum axle loads carried and the gross annual tonnage expected. In general the absolute minimum depth of ballast needed beneath sleepers for even a lightly loaded railway should never be less than 150mm and heavily loaded main lines can require as much as 280 mm. The currently recommended minimum thicknesses of ballast beneath sleepers for lines in the UK are as shown in

To ensure both lateral and longitudinal stability of the track, particularly when using continuously welded rail, it is essential that ballast is taken up to the level of the top of the sleepers between the sleepers and given a good ‗shoulder' at the sleeper ends. To achieve maintenance of this condition, regular inspection and periodic tamping is necessary.

Materials for Track Ballast

Good quality track ballast is made from crushed natural rock with particles not larger than 50mm nor generally smaller than 28mm. Angular stones are preferable to naturally rounded stones, to achieve the best interlock properties and resistance to longitudinal and lateral movement under dynamic loading. If ballast particles are larger than the maximum size stated there may only be two or three stones between the underside of the sleeper and the sub-grade which will be insufficient to properly distribute the load. Too

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many small stones below 28mm will however clog the ballast and reduce, in the longer term, its drainage properties. Samples of track ballast must be checked for grading by sieve analysis. Not more than 3% by weight should be retained on the 50mm square mesh sieve and not more than 2% should pass through the 28mm sieve.

Ballast particles can suffer degradation due to the action of traffic and maintenance operations in broadly two ways. Either edges can become rounded and lose their interlocking effect or particles can break or crush under repeated loading. Some of the softer stones suffer badly from attrition in the presence of water. This deterioration, particularly at rail joints, can be associated with ‗wet spots' in the track, which can cause rapid deterioration of line, level and riding comfort.

Certain tests can be introduced to check the wet attrition qualities of ballast. Generally speaking limestones tend to have poor wet attrition qualities, and crushed granite being one of the best, although expensive.

Sleeper Functions

Sleepers and bearers or timbers (for points and crossings) need to fulfil the following basic functions:

- Spread wheel loads to ballast.
- Hold rails to gauge and inclination.
- Transmit lateral and longitudinal forces.
- Insulate rails electrically.
- Provide a base for rail seats and fastenings.

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Sleepers are also often called upon to fulfil other secondary but important
functions which should not be overlooked. These include:
• Supporting wheels and/or jacks direct (in a derailment situation).
• Acting as transverse beams when sitting on temporary_waybeams‘.
• Supporting signal engineering and other safety related equipment such
as trip cocks and point motors.
• Supporting conductor rails, electrical bonds and feeder cables.
• Reducing noise and vibration on non-ballasted bridge decks.

Timber Sleepers

The traditional timber sleeper was accepted by most railways as standard
up to about the middle of the twentieth century, although its durability limitations were recognised.
Even today there are still many railways using timber sleepers, where the advantages of good resilience, ease of handling, adaptability to nonstandard
situations or electrical insulation are very important.
Timber sleepers and bearers for surface railways are usually made of softwood, either imported Douglas Fir or homegrown Scots Pine. The standard
dimensions for softwood sleepers used in the UK are 254mm wide by 127mm thick in cross-section by 2600mm long.
All softwoods used in sleepers and bearers must be thoroughly seasoned
and then impregnated under pressure with a suitable preservative before use. Traditionally this preservative has been hot creosote but other
materials have been used successfully in recent years which may have less
associated health hazards.
All lines in deep tube tunnels or in locations where fire could be a risk

are provided with sleepers and pit-blocks made from imported untreated
hardwood such as Jarrah.
Jarrah timbers used on the surface for points and crossing work which is not protected from the weather can last up to 35 years. In the protected
environment of dry tube tunnels, Jarrah sleepers on the London Underground
have been known to last in excess of fifty years before needing renewal.
The author has in his possession a handsome polished jarrah pen and pencil box which bears the following interesting inscription under the lid:
‗*This box is made from jarrah sleepers withdrawn from the London Tube railways after 54 years continuous service. It is estimated that during this time 500 million passengers travelled over the sleepers.*'
Such comment speaks for itself. Hardwood sleepers eventually usually
need replacing after this long period not because the general condition of
the timber has deteriorated but because it is by then not possible to get a
sound fixing for chair screws.
Softwood treated sleepers on the surface can be expected to last between
15 and 25 years depending on location and traffic use. Renewal is usually
required because bad splitting and/or rot has occurred.

Prestressed Concrete Sleepers (Monobloc)

As a substitute to softwood some experimental work was carried out during
the late 1930's on concrete sleepers. Originally ordinary reinforced concrete

was used but not found very satisfactory for a number of reasons. At that stage, concrete simply replaced timber, bullhead rails and cast iron chairs being used as in other conventional track.

After the Second World War prestressed concrete was developed and used extensively on new structures. The great advantage of prestressed concrete was that concrete is kept under compression under all conditions of flexure, both under load and after. This means that tension cracks do not occur which can allow the ingress of moisture and corrosion of embedded steel.

Development of prestressed sleepers took place about the same time as development of flat bottomed rail and direct fastenings.

At the time of writing the standard sleeper for main line railways in the UK is the F27(AS or BS) prestressed concrete sleeper manufactured by the pretensioned method. Variations of this standard sleeper are available with extra holes for supporting conductor rail insulators and with shallower depth where these conditions apply. In this method the prestressing tendons are tensioned prior to the concrete being placed and are only released once the concrete has reached sufficient compressive strength to resist the induced forces thus applied. This method is also sometimes referred to as the _Long Line' system, as sleepers are cast in long lines or beds of twenty

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five sleepers or more.
Some counties outside the UK adopt the post-tensioning method where
tendons are placed in debonding sheaths and the stress is applied after the concrete has hardened by application of tensile force to the tendons by
jacking and final anchoring. This method is slower but less capital intensive
and lends itself to small-scale production and situations where demand
is less.
Standard prestressed concrete sleepers used in the UK are normally 2515mm long by 264mm wide. The depth varies from 203mm at the rail
seat to 165mm at the centre line giving a total weight of 285 kg. The prestress
is provided by six No. 9.3mm strands for standard use increased to eight and strands for heavy duty. These sleepers are capable of sustaining
an equivalent dynamic load of 24 tonnes at each rail seat. Allowing for
impact, lurching, wheel flats, poor rail joints and etc this is equivalent to
the effects of the passage of a static 25 tonnes axle.
Metros and light rail systems have extensively also adopted prestressed
concrete sleepers. Where maximum axle loads are less than for main line as
shown above, the sleeper dimensions may be reduced accordingly. However,
great care must be taken in the design to ensure that adequate allowance
is made for dynamic effects, particularly for both ‗hogging' and ‗sagging'

bending moments.
The main disadvantage of the concrete sleeper over its timber predecessor
is that of weight. Timber sleepers were often manhandled into their final position and replacement of single defective sleepers (or _spotting‘ as
it is sometimes known) was also done by hand. With concrete sleepers some
form of mechanisation is required for these operations.

Twin Block Sleepers

The twin block sleeper consists of two reinforced concrete blocks joined
together with a steel tie bar cast into the blocks. This type of sleeper is used extensively in Europe, particularly in France, but not in the UK. The
standard sleeper weighs 230 kg which is less than the monobloc equivalent.
However handling and placing can be difficult due to the tendency to twist
when lifted. Twinblock sleepers can be provided with resilient _boots‘ and
can be incorporated into non-ballasted slab track or monolithic embedment
in road surfaces for light rail street running.

Steel Sleepers

Steel sleepers have been hardly ever used in the UK, largely because of cost
and fear of corrosion in our variable weather conditions. However, there are
countries throughout the world where these sleepers are used successfully,
particularly where trains run at moderate speeds only. Reference should be

made to BS 500. Most steel sleepers are inverted troughs which are either
rolled to that section or rolled flat and then hot pressed to the trough shape.
Being only 68 kg in weight, these sleepers are easy to handle but the inverted trough makes them difficult to satisfactorily pack with ballast.
They have been shown to be completely satisfactory however in sidings
and depots. Electrical insulation is necessary at fastenings if track circuits
are being used for train detection and this is not always a simple or effective
matter.
In some climates the normal coating of millscale and rust is sufficient to protect against significant loss of section by corrosion. Steel sleepers
can however be given protection by dipping in bitumen or oil during the
production process.

Rail Fastenings, Baseplates and Pads

Early railways adopted various forms of cast iron chair which were fixed
to the sleepers and in which rails sat, being held in position by hardwood
wedges or _keys'. All railways which used bullhead section rail used fixings
which were basically of this type. With the introduction of flat bottomed
rail starting in the late 1940's, a new form of fastening had to evolve.
The need was to design a resilient connection between rail and sleeper capable of resisting all forces induced by the passage of trains and by temperature
and weather variations over a long period of time.

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It was soon found that too rigid fixings became loose under vibration and that some degree of elasticity was necessary to resist both creep and
buckling. Maintenance of the clip clamping force on the rail foot or ‗toe
load' was soon realised as being of crucial importance in this respect.
Since the 1940's many FB rail resilient fasteners have been designed, manufactured and used throughout the world, with varying degrees of success.
These fastenings can be grouped into three distinctive types as shown below:

- An elastic rail spike. This is driven into pre-drilled holes in sleepers and
can be used with or without a steel or cast iron base plate.
- A spring clip bearing on the foot of the rail held down by a nut and bolt
element tightened to a predetermined torque. This type of fastening is still used widely in France and Germany.
- A spring clip driven into a hole or slot in a ‗shoulder', either cast into
the sleeper or part of a base plate. The act of driving in the clip either twists or bends the clip thus creating a toe load on the rail.

In the UK in recent years most railways, both main line and Metro, as well
as some light railways, have adopted the last type when using FB rail.
The standard fastening used by British Rail on all new FB track in recent years has been the ‗Pandrol' clip. This clip is made from circular
section spring steel bar by a process which involves heating the bar, hot
pressing into shape and then quenching and annealing. The majority of
plain line track on BR is laid on concrete sleepers without baseplates and in

this case the anchorage shoulder is cast into the sleeper during manufacture.

Where Pandrol rail clips are used in conjunction with base plates the latter are secured to the timber or sleeper by chair screws.

Where DC electrified railways have conductor rails running close to running rails, it is necessary to ensure that rail clips can be placed and maintained without potential damage or dislocation of the conductor rails and insulators. With the Pandrol clip this condition is satisfied as the clip is introduced into the shoulder and driven in a direction parallel to the running rail. Some earlier spring clips were driven at right angles to the rail which certainly would not be possible close to conductor rails.

Fastenings require insulation both from electrical current and from vibration/noise. This is achieved by the introduction of resilient insulating pads at points of contact.

Rails

All modern railways use steel rails which are specifically rolled for the purpose from steel which has the required qualities of strength, fatigue endurance and wear and corrosion resistance. This type of steel is fully covered by British Standard Specification 11.

As has been mentioned previously, the shape of the rail has now become generally standardised as the Flat Bottom (FB) rail. This is sometimes known as the Vignoles rail, after the inventor. Main line railways in the UK have now standardised on the BS113A section rail for all important lines.

The head of the rail has an almost flat top with curves at the outer

edges designed to fit the shape of the wheel tyre. One of the features of a
well matched rail head and wheel tyre is that, when the axis of the wheel
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set coincides with the longitudinal axis of the track and the rail is set at its correct inclination of 1 in 20 to the vertical, the point of contact between the two is very close to the centre line of the rail. This is very desirable since it minimises the twisting effect on the rail which a concentrically
applied wheel load would produce, and by keeping the contact
area away from the gauge corner, reduces both corner ‗shelling' and fatigue
damage.

The rail head sides slope at 1 in 20. This is to compensate for the 1 in 20
inwards slope of the rails and not only makes it simpler to check the gauge
but ensures that when side wear takes place the associated gauge widening
is minimised.

The thick web of the BS113A section is designed to give the rail adequate
shear strength to guard against fatigue failures, particularly around fishbolt
holes and under heavy axle loads at joints. The foot of the rail is broad enough to give stability against roll-over, remembering that steering forces
exerted by rolling stock produce torsional and lateral forces which have to
be resisted by the rail and transmitted via the fastenings to the sleeper. In
addition to the primary function, the rail has secondary functions relating

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to the carrying of track circuit currents and in some cases on electrified railways, conveying return traction currents.

Each section of rail that is used requires special steel castings, clips, bolts, resilient pads, fishplates, expansion switches and etc to make up the full structural system of the track. Most railway authorities endeavour to keep rail types and sizes to a minimum to ensure also that maintenance stocks of replacement components can also be kept to a sensible minimum.

A great deal of capital can be tied up in stock which is kept in stores just to cover an eventuality which may never happen.

There are also a number of signal related track components, like block joints, which are incorporated into the track structural system.

With third and fourth rail DC electrification systems there are also a large number of insulators and other fittings relating to the track which are required.

Rail Wear

Abrasive wear occurs when there is contact between the side of the flange of a wheel and the gauge face of the rail. This contact usually takes place between the leading outer wheel of a vehicle bogey and the outer rail of a curve.

On curves careful periodic check must be carried out of the outer rail to ensure that side wear is kept within prescribed limits. Failure to do this could result in a derailment. Where curves are tighter than 200 metre

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radius, continuous check rails should be provided inside the inner rail. This
check rail is to be set not more than about 50mm inside the running rail
or at a distance that will ensure that the inside face of the flange of the inner wheels will bear on the check rail thus sharing the centrifugal force
between the check rail and the outer rail through flange bearing.
Abrasive wear of rails can be reduced by the use of rail lubricators placed at strategic positions. Great care needs to be exercised in the use of
lubricators to ensure that only flanges are lubricated. Lubricant deposited
on the top of rail heads can cause problems with braking, acceleration and
wheel-spin. This is particularly important where trains are automatically
driven or where stopping positions are critical such as when rolling stock
doors have to line up with platform doors.
When wheels run along fairly straight track with flanges just clear of the rails, the contact area between wheel and rail is extremely small. In
theory the contact would only be a point which would make contact pressures
infinitely high. In practice both surfaces deform slightly to give a contact _patch‘. Even so, typically such a patch has only an area of about
100 sq.mm under the heaviest wheel load. This gives pressures as high as
1200N/sq.mm which is higher than the yield point of the steel. This has
the effect of causing the contact patch to become plastic and to flow causing

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various wear patterns and irregularities over time.
Where rails become side worn near to limit on curves, extra life can be obtained by either turning the high rail on jointed track or transposing the two rails on continuously welded rail. Close inspection of the existing inner rail outer edge must be carried out before transposing to ensure that there are no other defects present such as roll-over, ‗lipping' or plastic flow that would make the ride rough and precipitate failure of the new running edge.
If speeds in excess of 120 kph (75 mph) are expected, transposing should only be carried out if re-profiling of the existing inner rail is carried out.
Wear on point and crossings needs to be carefully watched on a regular basis. Some repair of bad wear can be done by welding but in most cases components need to be changed.
In jointed track excessive wear often takes place at rail joints or fishbolt holes and is the main reason for re -railing. Joints also increase wear on rolling stock. This is one of the main reasons why main line railways are progressively changing to continuously welded rails.
When a derailment occurs on any railway at any location, rail wear must be fully investigated as this can often prove to be the root cause. All rails should be closely inspected including any tell-tale signs of where wheels ran

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at the time of and just prior to the derailment.

Desirability of Removing Rail Joints

The earliest memories of many from childhood days relate to the _Clickerty-clack‘ of steam railways.

In those days every schoolboy knew that rails were sixty foot long and had to have fairly loose bolted joints so that the rails could expand in the hot weather and contract in the cold. Well understood also to the regular suburban commuter was the familiar sight from the carriage window of the plate-layer driving in keys and greasing fish bolts.

For many experienced railwaymen however, these _chores‘ represented a sizeable annual workload and removal of joints, if it could be done practically and safely, would be a giant leap forward. Apart from the reduction of potential track irregularities and smoothing and quietening down of the ride, removal of rail joints would clearly show a reduction of wear on wheels and rolling stock components in general. There would also be an improvement in the performance of under-frame and bogie components, which are highly subject to metal fatigue.

Up to the outbreak of the Second World War in 1939, mechanical, civil, structural and marine engineers had all used bolting and riveting as the main method of joining together steelwork in its various forms.

During the War, metal arc welding began to be used for the first time and after the War welding began to be used extensively, particularly in

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structures, machines and ships.

The Introduction of Track Welding

In the immediate post-war years, certain wartime teething troubles with metal arc welding were eventually ironed out and were better understood, as wider experience was gained. In particular, failure of welds or the parent metal in the heat affected zone of welds by metal fatigue took some time to be clearly understood and to be able to be correctly predicted. These fatigue failures were particularly troublesome in some of the early welded ships and to a lesser extent in some welded bridge members.

Metal arc welding was used extensively on steel structures in shop fabrication.

By the late fifties, shop welding of this type had completely replaced the earlier shop riveting of structures, site joints generally being site bolted or very infrequently, site welded.

Although some metal arc welding and electro-slag welding is used for the fabrication and repair of point and crossing work, the welding of rails end to end to form continuous welded rail (CWR) is carried out in the shops by a process known as Flash Butt Welding (FBW).

Flash butt welding of rails commenced in the UK on a large scale in the late 1950‘s and since that time the process has been refined and improved but still remains basically the same. In the mid-1950‘s London Underground introduced flash butt long welded rails using the standard bullhead section.

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The FBW rails were produced by welding five standard sixty foot lengths
into a long rail of 300 feet (about 90 metres). These rails were joined using
_tight' bolted joints where the fish-plates were clamped to the rail using high
strength friction grip bolts, tightened to a predetermined torque. London
Underground are now in the process of changing over to flat bottom rail.
Main line railways in the UK use flat bottom section rail for CWR which
is flash butt welded in the shops in lengths up to 240 metres. In recent years
in the UK British Steel PLC have been able to supply long lengths of rail
already flash butt welded into long lengths.

Shop Welding to Produce Long Rails

The process of Flash Butt Welding is used in the shops to join rails which
are later to be incorporated into ContinuousWelded Rail sites. This process
involves clamping the rails at a predetermined gap distance and passing a
high current across the gap at a low voltage, during which the work pieces
are brought together.
Electrical resistance heating first causes contacting surface irregularities
to melt and subsequently raises the temperature of the whole interface to
near melting point. Once the components are sufficiently heated they are

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forged together, and excess molten steel at the interface is forced out of the weld area.

The stages of FBW in the shops include burn off, preheating, flashing, forging and post weld treatment.

Once the weld has solidified, integral shears at the welding plant remove the excess upset from the periphery of the weld, leaving about 1mm proud all round the weld section. The welds are then straightened and the railhead ground to give a smooth profile for the weld along the length on the rail.

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Fig. 5.7. Continuous welded track.

Unlike with metal arc welding, no electrodes or added metal is used, only the parent metal is fused. Because some of the metal at the rail ends is forced out of the section profile, the overall effective length of the rail reduces by about 20mm for each weld.

5.16 Site Welding to Produce CWR

On arrival at site, long rails are welded to form CWR using the thermit or alumino thermic welding process. This method, which was discovered in 1896 by Hans Goldmidt, is based on the reduction of heavy metal oxides by aluminium. Thermit welding was first used in Hungary in 1904 and most of Europe had adopted the process for site rail joints by the late 1920s. The process was not used very widely in the UK however, until the 1950‘s.

Some light railways have used thermit welding of short rails throughout, without the use of FBW into long rails beforehand. Although this is cheaper and removes the need for a shop process, the practice is not recommended for railways carrying heavy axle loads. Thermit welds are completely satisfactory but have less consistency than FBW, being carried out in the open on site rather than in controlled workshop conditions.

Annual statistics, published on reported broken rails at welds in the UK over recent years, strongly bear out the better performance of FBW in practice.

In this process the rails to be joined are set in position, fixed in their baseplates, with the ends properly aligned and with a gap between them of between 22 and 26mm. A refractory mould is then placed around the joint and a thermit portion is ignited in a refractory crucible above the mould.

The portion is a combination of powders which after reaction will produce a weld metal which matches the chemistry and metallurgy of the parent rails. When the reaction is complete the crucible is tapped and steel pours into the moulds to form the weld. Slag, being less dense than the steel, remains at the top of the mould. The weld is allowed to cool after which the excess metal, mould material and slag is trimmed away and the joint is ground to profile.

Stressing or 'Locking-up' of CWR

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With jointed short rails, the object is to allow rails to expand and contract during extremes of temperature to avoid the build up of compressive and tensile stresses. In long welded rails and CWR however, the rail is constrained so that it cannot expand or contract. In this case, in order that the rail shall remain at its original length, the rail undergoes compressive and tensile strain, which is equal and opposite to thermal strain.

By simple calculation using Hooke's Law (F = *strain* × A × E) *it can be seen that a restrained standard BS113A FB rail increased in temperature, by say 45◦C, will produce a force of 76.5 tonnes in the rail.*

A compressive force of such magnitude in hot weather is sufficient to cause a buckle of the track and it is essential for safety that development of such a force is prevented. Similarly, high tensile forces in extremely cold weather can cause brittle fracture of rails and must be avoided. This is done on CWR by artificially extending the rail at the time of installation and fixing it down in a state of tension. The ideal is to fix the rail at a length that it will be at a temperature that is exactly halfway between the hottest and coldest likely rail temperature. In the UK this is generally accepted as a temperature of 27◦ C.

The rail may be artificially extended by rail warming or, as is now more usual, by stretching with a tensor.

5.18 Points, Switches and Crossings

All railways require points or ‗turnouts' to be able to divert trains from one

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track to another and crossings or _diamonds‘ to allow trains to cross other
tracks at an angle.
This applies to all railways from the most complicated reversible layouts
at terminal stations to simple single track tramways with passing loops.
Any assembly of points and crossings is called a layout. Some layouts occur
frequently and have acquired their own names. The most common is the
_crossover‘ which is simply two sets of points laid crossing to crossing in
adjoining track enabling trains to change track in one direction. If two crossovers are superimposed, thus enabling movements from either track
in either direction, the layout is known as a _scissors crossover‘ for obvious
reasons. In this layout there are four sets of points and one central diamond. Points or turnouts and diamonds are themselves composed of
elements known as crossings and switches.

Crossing Design and Manufacture

A crossing enables a wheel travelling along a given rail to pass through the
rail of a track which crosses its path. Where two tracks cross each other
at an angle there are four crossings which make up the resulting diamond.
Unless the tracks cross at right angles, there will be two Obtuse Crossings
and two at an acute angle known as Common Crossings.
_Built-up‘ Crossings are manufactured from standard rail and are perhaps

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the most often seen, having been used traditionally on railways for many decades. In these crossings the four components, the point rail, the splice rail and two wing rails, are cut, bent to shape, drilled and machined as necessary and then bolted together as a complete assembly. This type of simple crossing has given good service over many years in countries all round the world. They are subject to wear however, particularly at the tip of the point rail and where the point and splice rail bear against one another. Through bolts also often work loose under traffic.

A ‗part-welded' crossing consists essentially of the same four rails as a built-up crossing and is usually made of standard rail. The assembly however is strong enough to take thermal loads and consequently it can be welded into CWR, leaving only the flange-way gap as a source of wheel/rail impact. In theory at least, this is a considerable advantage over both builtup crossings and cast crossings, although ‗welding-in' of components into point and crossing layouts can have a significant time disadvantage when work becomes necessary during possession. There are also disadvantages when one element fails undertraffic and has to be cut out and re-welded rather than re-bolted into position.

The ‗Vee' of a part-welded crossing is prepared by machining two pieces of rail into a symmetrical straight splice with a weld preparation milled

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into the head and foot. The electro-slag welding process is used under carefully
controlled conditions to produce a continuous homogeneous weld. This
welding is laid down automatically with top and bottom welds being done
simultaneously to keep any distortion to an absolute minimum. The complete
crossing assembly is held together using high strength friction grip
bolts tightened to a specified torque or by _huck' bolts.
Another form of crossing is the cast Austenitic Manganese Steel (or
AMS) crossing. In this case there is only one _monobloc' component making
up the entire casting. The casting is made by pouring this special molten
steel into a mould which represents the shape of all four components used
in the other types of crossing. This type of crossing is favoured by many
railways due to its very high wear resistance and long life. Also due to
being monolithic there is no relative movement of components and the ride
is generally very good. Another advantage is the ability to combine more
than one crossing in a single casting, as is sometimes the case on a tight
scissors crossover.
In spite of its advantages however, AMS crossings do have some disadvantages.
Casting as a process is always subject to internal cracking due
to cooling and these faults are sometimes difficult to detect before installation.
Also when faults do arise in service, the castings are much heavier

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and more unwieldy to handle during a limited possession than built-up crossings, particularly in tunnel.

Check rails are provided opposite crossings. Their function is to control the alignment of the wheel-set so that it is not possible for the wheel moving across the gap in the throat of the crossing to strike the nose of the crossing or to take a wrong path.

Points or Turnouts

Points or turnouts, as shown in Fig. 5.9, enable vehicles to be diverted from one track to another and consist of a pair of switches and a crossing, connected by closure rails.

In a set of points the fixed rails on either side are known as stock rails, the moveable rail being known as the switch rail. The switch rail is machined to a sharp tip or toe at one end and the tapered portion of the switch rail is known as the switch tongue. The switch tongue is machined to fit snugly into the stock rail in the workshops. It is unwise when worn to change a stock or switch on its own and both should be changed as a fitting pair.

Two movable switches should be held in the correct relative position to each other by at least two stretcher bars.

If the set of points is so arranged that in the predominating traffic direction the tracks diverge, it is known as facing points. If the main traffic direction is such that the two lines merge, they are trailing points.

Driving, Locking and Detection of Points

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In the early days of railways, sets of facing points on passenger lines were
avoided because of the high risk of derailment due to wheel flanges ‗splitting'
stocks and switches. Following this early experience, it became mandatory
that all facing points should be locked in position and that the position of
each switch should be ‗detected' in relation to its mating stock rail.
On modern railways, points are operated by electric or compressed air
point motors/machines which operate the points, lock and clamp them in
position and also detect whether or not the switches are fully ‗home'. There
needs to be careful and clear division of responsibility for maintenance and
adjustment of all point mechanisms between signal and track engineers.

Conductor Rails and Components

Where railways are electrified using either third rail or fourth rail DC systems
there are a number of other components and fittings which are track related.
Conductor rails are usually made from steel which is designed to be of
high electrical conductivity, containing much less carbon than for normal
rails. This means that steel conductor rails are softer and of lower strength
than running rails. The rails can be jointed by bolted fish plates or welded.
In recent years, some light rail systems (e.g. DLR.) have used Aluminium

conductor rails for underside contact, with a wearing surface of stainless
steel.
Conductor rails are supported by insulators fixed to sleepers at frequencies
depending on track curvature, location and type of fixing. The insulator
assembly usually consists of a porcelain pot with a cast malleable iron cap
having two upstanding ‗ears‘. These ears locate the conductor rail transversely
without restraining longitudinal movement. The insulators are
fixed to the sleeper using a pair of wrap round base clips.
At discontinuities and ends of conductor rails, ramps are provided, also
supported on sleepers, to pick up and lower collector shoes on rolling stock.
It is important that these ramps, which can be welded steel or cast iron,
are regularly checked to ensure that line and level is correct. Failure to do
this can result in damage to rolling stock or track or both.

Paved Concrete Track

Paved Concrete Track (sometimes known as PACT) is a continuously reinforced
concrete pavement laid by a specially designed ‗slip-form‘ paver.
This machine runs on accurately positioned guide rails which ensure that
the concrete pavement line and level is very closely controlled. The guide
rails are often the long welded rails which will subsequently be repositioned
and used as permanent running rails.
The rails are usually supported on base-plates which may have some

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form of resilience incorporated into their design. Even though the concrete has been accurately positioned, the tolerances achieved may be more than is desirable for accurate positioning of the rails. It is desirable therefore that some adjustment capability is built into the system of final positioning of the base-plates or cast-in fixings. One way of achieving this is for the rails to be finally positioned to line and level on temporary packs/wedges with base-plates and fixing bolts hanging off the rail. Once final rail position is fixed, any gaps at fixing holes and under base-plates can then be grouted up or filled using epoxy mortar.

This track system is much more expensive than conventional ballasted track and cannot be easily modified once laid. It is however of particular use in existing main line size tunnels, where the shallow construction depth may permit the achievement of increased overhead clearances for 25 kV electrification or for the passage of large container trains. In this track system particular attention needs to be given to drainage channels.

Cast-in Precast Sleeper Track

As a cheaper alternative to PACT, prestressed or reinforced concrete sleepers or special purpose made units can be laid in position accurately with rails fully adjusted and then a concrete slab poured between and around

them. In this case, holes through the sleepers are left for transverse reinforcement
or some _hedgehog' starter bars are provided to assist both
the precast and *in-situ* elements to act as a whole.

Floating Slab Track

In locations where it is vitally important to reduce noise and vibration to
an absolute minimum, floating slab track may be considered. It should be
stressed that this type of solution is very expensive, requires a lot of space
and can only be justified where railways run very close to or under concert
halls, churches or operating theatres, etc.
In this form of construction the track, which may be ballasted or nonballasted,
is supported on a structure which is isolated from the supporting
ground by soft resilient bearings.
A notable example of this type of construction is to be found in London
under the new Barbican Concert Hall.

5.26 Track Installation and Renewal

Up to the late 1930's, most railways installed or renewed track mainly
_piece-small', using a large amount of skilled labour, only assisted for heavy
lifts by rail mounted steam cranes.
In more recent years special _purpose-built' equipment has been produced,
in particular for surface main lines which mechanises much of the
track laying process. Large machines can now lay panels of sleepered track
or place individual sleepers, to be followed by plant laying welded rails in
very long lengths.

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Because of space restrictions in tube and other small bore tunnels, much of the laying of tracks in these tunnels is still carried out piece-small, using manual methods but using power tools and aids wherever possible. This has the added complication in tube tunnels that night possessions for renewal work are short and track has to be made safe each morning for a further day's running.

5.27 Day-to-day Maintenance of Track

The passage of trains coupled with the effects of varied weather and day/night conditions, causes steady deterioration of even the best constructed railway track unless proper day-to-day maintenance is carried out.

Activities of others alongside the railway and trespassers and vandals on the railway can also effect track conditions of safety.

Both visual inspection of condition and mechanical measurement of track geometry is necessary to establish a quality standard and to determine whether the standard is being maintained or not.

All railways require a track maintenance organisation to ensure adequate inspection is carried out and that proper resources are available to attend to minor matters on the track and immediate surroundings as they arise.

On surface lines, where it is possible to safely stand to one side to allow trains to pass, much daily inspection and local adjustment can be carried out during traffic hours.

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On underground railways or other urban railways where clearances are
tight and trains are frequent, access for staff is not usually available during
traffic hours. In this case maintenance staff must be organised to be on
duty at night during non-traffic hours. For these railways all inspection
and adjustment of track must be done at night and cannot be watched or
further adjusted during the following day except under special protection
arrangements which will inevitably delay trains.
Regular major maintenance activities which will obstruct traffic or
endanger staff, need to be arranged during non-traffic hours or in a _possession‘
of the track specifically arranged for the purpose.
Such major activities might well include ballast tamping, drain rodding,
rubbish clearing, block joint changing, fence repairs close to the track and
replacing individual damaged sleepers, chair castings or rails

AirportPlanningStudies

Introduction

The planning of an airport is such a complex process that the analysis of one activity without regard to the effect on other activities will not provide acceptable solutions. An airport encompasses a wide range

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of activities which have different and often conflicting requirements. Yet they are interdependent so that a single activity may limit the capacity of the entire complex. In the past airport master plans were developed on the basis of local aviation needs. In more recent times these plans have been integrated into an airport system plan which assessed not only the needs at a specific airport site but also the overall
needs of the system of airports which service an area, region, state, or country. If future airport planning efforts are to be successful, they must be founded on guidelines established on the basis of comprehensive
airport system and master plans.
The elements of a large airport are shown in Fig. 4-1. It is divided into two major components, the airside and the landside. The aircraft gates at the terminal buildings form the division between the two components. Within the system, the characteristics of the vehicles, both ground and air, have a large influence on planning. The passenger
and shipper of goods are interested primarily in the overall doorto-door travel time and not just the duration of the air journey. For this reason access to airports is an essential consideration in planning. The problems resulting from the incorporation of airport operations into the web of metropolitan life are complex. In the early days of air transport, airports were located at a distance from the city, where inexpensive land and a limited number of obstructions permitted
flexibility in airport operations. Because of the nature of aircraft and the infrequency of flights, noise was not a problem to the

Types of Studies

Many different types of studies are performed in airport planning. These include studies related to facility planning, financial planning, traffic and markets, economics, and the environment. However, each of these studies can usually be classified as being performed at one of three levels: the system planning level, the master planning level, or

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the project planning level.

The Airport System Plan

An airport system plan is a representation of the aviation facilities required to meet the immediate and future needs of a metropolitan area, region, state, or country. The National Plan of Integrated Airport Systems (NPIAS) [11] is an example of a system plan representing the
airport development needs of the United States. The Michigan Aviation
System Plan [10] is an example of a system plan representing the airport development needs of the state of Michigan, and the Southeast Michigan Regional Aviation System Plan [13] is a system plan representing the airport development needs of a seven county region comprising the Detroit Metropolitan area.

The system plan presents the recommendations for the general location and characteristics of new airports and heliports and the nature of expansion for existing ones to meet forecasts of aggregate demand. It identifies the aviation role of existing and recommended new airports and facilities. It includes the timing and estimated costs of development and relates airport system planning to the policy and objectives of the relevant jurisdiction. Its overall purpose is to determine
the extent, type, nature, location, and timing of airport development needed to establish a viable, balanced, and integrated system of airports

It also provides the basis for detailed airport planning
such as that contained in the airport master plan.

The airport system plan provides both broad and specific policies, plans, and programs required to establish a viable and integrated system of airports to meet the needs of the region. The objectives of the system plan include

1. The orderly and timely development of a system of airports adequate to meet present and future aviation needs and to promote the desired pattern of regional growth relative to

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industrial, employment, social, environmental, and recreational goals.

2. The development of aviation to meet its role in a balanced and multimodal transportation system to foster the overall goals of the area as reflected in the transportation system plan and comprehensive development plan.

3. The protection and enhancement of the environment through the location and expansion of aviation facilities in a manner which avoids ecological and environmental impairment.

4. The provision of the framework within which specific airport programs may be developed consistent with the short- and long-range airport system requirements.

5. The implementation of land-use and airspace plans which optimize these resources in an often constrained environment.

6. The development of long-range fiscal plans and the establishment of priorities for airport financing within the governmental budgeting process.

7. The establishment of the mechanism for the implementation of the system plan through the normal political framework, including the necessary coordination between governmental agencies, the involvement of both public and private aviation and nonaviation interests, and compatibility with the content, standards, and criteria of existing legislation.

The airport system planning process must be consistent with state, regional, or national goals for transportation, land use, and the environment. The elements in a typical airport system planning process
[8] include the following:

1. Exploration of issues that impact aviation in the study area
2. Inventory of the current system
3. Identification of air transportation needs
4. Forecast of system demand
5. Consideration of alternative airport systems

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6. Definition of airport roles and policy strategies
7. Recommendation of system changes, funding strategies, and airport development
8. Preparation of an implementation plan

Although the process involves many varied elements, the final product will result in the identification, preservation, and enhancement of the aviation system to meet current and future demand. The ultimate result of the process will be the establishment of a viable, balanced, and integrated system of airports.

Airport Site Selection

The emphasis in airport planning is normally on the expansion and improvement of existing airports. However if an existing airport cannot be expanded to meet the future demand or the need for a new airport is identified in an airport system plan, a process to select a new airport site may be required. The scope of the site selection process will vary with size, complexity, and role of the new airport, but there are basically three steps—identification, screening, and selection.

Identification—criteria is developed that will be used to evaluate different sites and determine if a site can function as an airport and meets the needs of the community and users. One criterion will be to identify the land area and basic facility requirements for the new airport. Part of this analysis will be a definition of airport roles if more than two airports serve the region. Other criteria might be that sites are within a certain radius or distance from the existing airport or community, or that sites should be relatively flat. Several potential sites that meet the criteria are identified. *Screening* — once sites are identifi ed, a screening process can be applied to each site. An evaluation of all potential sites that meet the initial criteria should be conducted, screening out those with the most obvious shortcomings. Screening factors might include topography, natural and man-made obstructions, airspace, access,

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environmental impacts, and development costs. If any sites are eliminated from further consideration, thorough documentation of the reasons for that decision is recommended. The remaining potential sites should then undergo a detailed comparison using comprehensive evaluation criteria. While the criteria will vary, the following is typically considered:

O*perational capability*—airspace considerations, obstructions, weather

Capacity potential—available land, suitability for construction, Weather

Ground access—distance from the demand for aviation services, regional highway infrastructure, available public transportation modes

Development costs—terrain, land costs, land values, soil conditions, availability of utilities

Environmental consequences—aircraft noise, air quality, groundwater runoff, impact on fl ora and fauna, existence of endangered species or cultural artifacts, historical features, changes in local land use, relocation of families and businesses, changes in socioeconomic characteristics

Compatibility with area-wide planning—impact on land use, effect on comprehensive land-use plans and transportation plans at the local and regional levels

Selection—the fi nal step is selecting and recommending a preferred site. While a weighting of the evaluation criteria and weighted ratings or ranking of the alternative sites is often used in selecting a site, caution must be used in applying this technique since it introduces an element of sensitivity into the analysis. The process should focus on providing decision makers with information on the various sites in a manner that is understandable and unbiased.

The Airport Master Plan

An airport master plan is a concept of the ultimate development of a specific airport. The term development includes the entire airport

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area, both for aviation and nonaviation uses, and the use of land adjacent to the airport [1, 4, 9]. It presents the development concept graphically and contains the data and rationale upon which the plan is based. Figure 4-2 shows a simple flowchart of the steps for preparing

an airport master plan. Master plans are prepared to support expansion and modernization of existing airports and guide the development of new airports.

The overall objective of the airport master plan is to provide guidelines for future development which will satisfy aviation demand in a financially feasible manner and be compatible with the environment, community development, and other modes of transportation. More specifically it is a guide for

1. Developing the physical facilities of an airport
2. Developing land on and adjacent to the airport
3. Determining the environmental effects of airport construction and operations
4. Establishing access requirements
5. Establishing the technical, economic and financial feasibility of proposed developments through a thorough investigation of alternative concepts
6. Establishing a schedule of priorities and phasing for the improvements proposed in the plan
7. Establishing an achievable financial plan to support the implementation schedule
8. Establishing a continuing planning process which will monitor conditions and adjust plan recommendations as circumstances warrant

Guidelines for completing an airport master plan are described by ICAO [4] and in the United States by the FAA [1]. A master plan report is typically organized as follows:

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Master plan vision, goals, and objectives—establishes the vision and overarching goals for the master plan as well as objectives thatwill guide the planning process and help ensure that the goals are achieved and the vision is realized.

Inventory of existing conditions—provides an overview of the airport's history, role in the region and nation, growth and development over time, description of its physical assets (airfi eld and airspace, terminal, ground access, and support facilities), and key industry trends.

Forecast of aviation demand—future levels of aircraft operations, number of passengers, and volume of cargo are forecasted for short, intermediate, and long-range time periods. Typically forecasts are made for 5, 10, and 20 years on both annual as well daily and busiest hours of the day.

Demand/capacity analysis and facility requirements—compares the future demand with the existing capacity of each airport component and identifi es the facility requirements necessary to accommodate the demand.

Alternatives development—identifi es, refi nes, and evaluates a range of alternatives for accommodating facility requirements. If the existing site cannot accommodate the anticipated growth, a selection process to fi nd a new site may be necessary.

Preferred development plan—identifi es, describes, and defi nes the alternative that best achieves the master plan goals and objectives. Figure 4-3 illustrates the development plan for the Chicago O'Hare International Airport.

Implementation plan—provides a comprehensive plan for the implementation of the preferred development plan, including the defi nition of projects, construction sequence and timeline, cost estimates, and fi nancial plan.

Environmental overview—provides an overview of the anticipated environmental impacts associated with the preferred development

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plan in order to understand the severity and to help expedite subsequent environmental processing at the project specifi c stage.

Airport plans package—documents that show the existing as well as planned modifi cations are prepared and the more notable is the airport layout plan (ALP). It comprises drawings that include the airfi eld's physical facilities, obstruction clearance and runway approach profi les, land -use plans, terminal area and ground access plans, and a property map. Specifi c guidelines for the airport layout plan in the United States are identifi ed by FAA [1].

Stakeholder and public involvement—documents the coordination efforts that occur among the stakeholders throughout the study.

The Airport Project Plan

A project plan focuses on a specific element of the airport master plan which is to be implemented in the short term and may include such items as the addition of a new runway, the modification of existing of runways, the provision of taxiways or taxiway exits, the addition of gates, the addition to or the renovation of terminal building facilities, or the modification of ground access facilities

The overall objective of the airport project plan is to provide the specific details of the development which will satisfy immediate aviation needs and be consistent with the objectives and constraints identified in the airport master plan. More specifically it is a detailed plan for

1. Developing the specific physical facilities at an airport including the architectural and engineering design for these facilities
2. Determining the environmental effects of this development through the construction and operational phases
3. Determining the detailed costs and financial planning for the

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development
4. Establishing a schedule for the construction and phasing of the specific items of development in the plan

Land-Use Planning

A land-use plan for property within the airport boundary and in areas adjacent to the airport is an essential part of an airport master plan. The land-use plan on and off the airport is an integral part of an area wide comprehensive planning program, and therefore it must be coordinated with the objectives, policies, and programs for the area which the airport is to serve. Incompatibility of the airport with its neighbors stems primarily from the objections of people to aircraft noise. A land-use plan must therefore project the extent of aircraft noise
that will be generated by airport operations in the future. Contours of equal intensity of noise can be drawn and overlaid on a land-use map and from these contours an estimate can be made of the compatibility of existing land use with airport operations. If the land outside the airport is underdeveloped, the contours are the basis for establishing comprehensive land-use zoning requirements.

Although zoning is used as a method for controlling land use adjacent to an airport, it is not effective in areas which are already built-up because it is usually not retroactive. Furthermore jurisdictions having zoning powers may not take effective zoning action.

Aircraft operations into and out of the airport may be made unnecessarily
complex to minimize noise encroachment on incompatible land uses. Despite these shortcomings the planner should utilize zoning as a vehicle to achieve compatibility wherever this approach is feasible.

Airports become involved in two types of zoning. One type is height and hazard zoning, which is mainly to protect the approaches to the airport from obstructions. The other type is land-use zoning. The extent of land use in the airport depends a great deal on the

amount of acreage available. Land uses can be classified as eitherclosely related to aviation or remotely related to aviation. Those closely related to aviation use include the runways, taxiways, aprons, terminal buildings, parking, and maintenance facilities. Nonaviation uses include space for recreational, industrial, and commercial activities. When considering commercial or industrial activities, care should be taken to ensure that they will not interfere with aircraft operations, communications equipment, and aids to navigation on the ground. Recreational facilities such as golf courses may be suitable within the immediate proximity of the airport boundary or certain agricultural uses are also appropriate as long as they do not attract birds. When there is acreage within the airport boundary in excess of aviation needs, it is sound fiscal planning to provide the greatest financial return from leases of the excess property.

Thus the land-use plan within the airport is a very effective tool in helping airport management make decisions concerning requests for land use by various interests and often airports delineate areas on the airport property for the development of industrial parks.

The principal objective of the land- use plan for areas outside the airport boundary is to minimize the disturbing effects of noise. As stated earlier the delineation of noise contours is the most promising approach for establishing noise-sensitive areas. The contours define the areas which are or are not suitable for residential use or other use and, likewise, those which are suitable for light industrial, commercial,

or recreational activity. Although the responsibility for developing land uses adjacent to the airport lies with the governing bodies of adjacent communities, the land -use plan provided by the airport authority will greatly influence and assist the governing bodies in their task of establishing comprehensive land-use zoning.

Environmental Impact Assessment

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Environmental factors must be considered carefully in the development of a new airport or the expansion of an existing one. In the United States, this is a requirement of the Airport and Airway Improvement Act of 1982 and the Environmental Policy Act of 1969. Studies of the impact of the construction and operation of a new airport or the expansion of an existing one upon acceptable levels of air and water quality, noise levels, ecological processes, and demographic development of the region must be conducted to determine how the airport requirements can best be met with minimal adverse environmental and social consequences.

Aircraft noise is the severest environmental problem to be considered in the development of airport facilities. Much has been done to quiet engines and modify flight procedures, resulting in substantial reductions in noise. Another effective means for reducing noise is through proper planning of land use for areas adjacent to the airport. For an existing airport this may be difficult as the land may havealready been built up. Every effort should be made to orient air traffic away from noise-sensitive land development.

Other important environmental factors include air and water pollution, industrial wastes and domestic sewage originating at the airport, and the disturbance of natural environmental values. In regard to air pollution, the federal government and industry have worked jointly toward alleviating the problem, and there is a reason to believe that it will probably be eliminated in the near future as an environmental factor. An airport can be a major contributor to water pollution if suitable treatment facilities for airport wastes are not provided. Chemicals used to deice aircraft are a major source of potential ground water pollution and provisions need to be made to safely dispose of this waste product. The environmental study must

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include a statement detailing the methods for handling sources of water pollution.

The construction of a new airport or the expansion of an existing one may have major impacts on the natural environment. This is particularly true for large developments where streams and major drainage courses may be changed, the habitats of wildlife may be disrupted, and wilderness and recreational areas may be reshaped. The environmental study should indicate how these disruptions might be alleviated.

In the preparation of an environmental study, or an environmental impact statement, the findings must include the following items [12]:

1. The environmental impact of the proposed development
2. Any adverse environmental effects which cannot be avoided should the development be implemented
3. Alternatives to the proposed development
4. The relationship between local short-term uses of the environment and the maintenance and enhancement of long-term productivity
5. Any irreversible environmental and irretrievable commitments of resources which would be involved in the proposed development should it be implemented
6. Growth inducing impact
7. Mitigation measures to minimize impact

In the application of these guidelines attention must be directed to the following questions. Will the proposed development

1. Cause controversy
2. Noticeably affect the ambient noise level for a significant number of people
3. Displace a significant number of people
4. Have a significant aesthetic or visual effect
5. Divide or disrupt an established community or divide existing uses

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6. Have any effect on areas of unique interest or scenic beauty
7. Destroy or derogate important recreational areas
8. Substantially alter the pattern of behavior for a species
9. Interfere with important wildlife breeding, nesting, or feeding grounds
10. Significantly increase air or water pollution
11. Adversely affect the water table of an area
12. Cause excessive congestion on existing ground transportation facilities
13. Adversely affect the land-use plan for the region

The preparation of an environmental impact statement based upon an environmental assessment study is an extremely important part of the airport planning process. The statement should clearly identify the problems that will affect environmental quality and the proposed actions to alleviate them. Unless the statement is sufficiently comprehensive, the entire airport development may be in jeopardy.

Economic and Financial Feasibility

The economic and financial feasibility of alternative plans for a new airport or expansion of an existing site must be clearly demonstrated by the planner. Even if the selected alternative is shown to be economically
feasible, then also it is necessary to show that the plan will
generate sufficient revenues to cover annual costs of capital investment,
administration, operations, and maintenance. This must be determined for each stage or phase of development detailed in the airport master plan.

An evaluation of economic feasibility requires an analysis of benefits and costs. A comparison of benefits and costs of potential capital investment programs indicates the desirability of a project from an economic point of view. The economic criterion used in evaluating an aviation investment is the total cost of facilities, including quantifiable
social costs, compared with the value of the increased effectiveness

measured in terms of total benefits. The costs include capital investment, administration, operation, maintenance, and any other costs that can be quantified. The benefits include a reduction in aircraft
and passenger delays, improved operating efficiency, and other benefits. The costs and benefits are usually determined on an annual basis.

Airport Classification

For the purpose of stipulating geometric design standards for the various types of airports and the functions which they serve, letter and numerical codes and other descriptors have been adopted to classify airports.

For design purposes, airports are classified based on the aircraft they accommodate. While at any airport, a wide variety of aircraft, from small general aviation piston-engine aircraft to heavy air transport

aircraft, will use the airfield, airports are designed based on a series of ―critical‖ or ―design‖ aircraft. These aircraft are selected from
the fleet using the airport as those most critical to airfield design. The FAA defines the term *critical aircraft* as the aircraft most demanding on airport design that operates at least 500 annual itinerant operations at a given airport. In many cases, more than one critical aircraft will be selected at an airport for design purposes. For example, it is often the smallest aircraft that is critical to the orientation of runways, while the largest aircraft determines most of the other dimensional specifications of an airfield.

The airport reference code is a coding system used to relate the airport design criteria to the operational and physical characteristics of the aircraft intended to operate at the airport.

It is based upon the *aircraft approach category* and the *airplane design group* to which the aircraft is assigned. The aircraft approach category, is determined by the aircraft

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approach speed, which is defined as 1.3 times the stall speed in the landing configuration of aircraft at maximum certified landing weight [6].

The airplane design group (ADG) is a grouping of aircraft based upon wingspan or tail height, An airplane design group for a particular aircraft is assigned based on the greater (higher Roman numeral) of that associated with the aircraft‘s wingspan or tail height.

The airport reference code is a two designator code referring to the aircraft approach category and the airplane design group for which the airport has been designed. For example, an airport reference code of B-III is an airport designed to accommodate aircraft with approach speeds from 91 to less than 121 kn (aircraft approach category B) with wingspans from 79 to less than 118 ft or tail heights from 30 to less than 45 ft (airplane design group III). The FAA publishes a list of the airport reference codes for various aircraft in Advisory Circular 150/5300-13 ―Airport Design‖ [6].

As an example, an airport designed to accommodate the Boeing 767-200 which has an approach speed of 130 kn (aircraft approach category C) and a wingspan of 156 ft 1 in (airplane design group IV) would be classified with an airport reference code C-IV.

The ICAO uses a two- element code, *the aerodrome reference code*, to classify the geometric design standards at an airport [2, 3]. The code elements consist of a numeric and alphabetic designator. The aerodrome code numbers 1 through 4 classify the length of the runway available, the *reference field length*, which includes the runway length and, if present, the stopway and clearway. The reference field length is the approximate required runway takeoff length converted to an equivalent length at mean sea level, 15°C, and zero percent gradient.

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The aerodrome code letters A through E classify the wingspan and outer main gear wheel span for the aircraft for which the airport has been designed.

1 kn is approximately 1.15 mi/h

Utility Airports

A*utility airport* is defined as one which has been designed, constructed, and maintained to accommodate approach category A and B aircraft [6].

The specifications for utility airports are grouped for *small aircraft*, those of maximum certified takeoff weights of 12,500 lb or less, and *large aircraft*, those with maximum certified takeoff weight in excess of 12,500 lb.

Design specifications for utility airports are governed by the airplane design group and the types of approaches authorized for the airport runway, that is, visual, nonprecision instrument or precision instrument approaches.

Utility airports for small aircraft are called *basic utility stage I*, *basic utility stage II*, and *general utility stage I*. Utility airports for large aircraft

are called *general utility stage II*. Utility airports are further grouped for either visual and nonprecision instrument operations or precision instrument operations. The visual and nonprecision instrument operation utility airports are the basic utility stage I, basic utility stage II, or general utility stage I airports. The precision instrument operation utility airport is the general utility stage II airport.

A basic utility stage I airport has the capability of accommodating about 75 percent of the single engine and small twin engine aircraft used for personal and business purposes. This generally means aircraft

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weighing on the order of 3000 lb or less is given the airport reference code B-I, which indicates that it accommodates aircraft in aircraft approach categories A and B and aircraft in airplane design group I. A basic utility stage II airport has the capability of accommodating all of the airplanes of a basic utility stage I airport plus some small business and air taxi-type airplanes. This generally means aircraft weighing on the order of 8000 lb or less is also given the airport reference

code B- I. A general utility stage I airport accommodates all small aircraft. It is assigned the airport reference code of B-II. A general utility stage II airport serves large airplanes in aircraft approach categories A and B and usually has the capability for precision instrument operations.

It is assigned the airport reference code of B-III.

Transport Airports

A *transport airport* is defined as an airport which is designed, constructed, and maintained to accommodate aircraft in approach categories C, D, and E [6]. The design specifications of transport airports are based upon the airplane design group.

Runways

A runway is a rectangular area on the airport surface prepared for the takeoff and landing of aircraft. An airport may have one runway or several runways which are sited, oriented, and configured in a manner to provide for the safe and efficient use of the airport under a variety of conditions. Several of the factors which affect the location, orientation, and number of runways at an airport include local weather conditions, particularly wind distribution and visibility, the topography of the airport and surrounding area, the type and amount of air traffic to be serviced at the airport, aircraft performance requirements, and aircraft noise [2].

Runway Configurations

The term ―runway configuration‖ refers to the number and relative orientations of one or more runways on an airfield. Many runway configurations exist. Most configurations are combinations of several basic configurations. The basic configurations are (1) single runways, (2) parallel runways, (3) intersecting runways, and (4) open-V runways.

Single Runway

It has been estimated that the hourly capacity of a single runway in VFR conditions is somewhere between 50 and 100 operations per hour, while in IFR conditions this capacity is reduced to 50 to 70 operations per hour, depending on the composition of the aircraft mix and navigational aids available [4].

Parallel Runways

The capacities of parallel runway systems depend on the number of runways and on the spacing between the runways. Two, three, and four parallel runways are common. The spacing between parallel runways varies widely. For the purpose of this discussion, the spacing is classified as close, intermediate, and far, depending on the centerline separation between two parallel runways. Close parallel runways are spaced from a minimum of 700 ft (for air carrier airports) to less than 2500 ft [5]. In IFR conditions an operation of one runway is dependent on the operation of other runway. Intermediate parallel runways are spaced between 2500 ft to less than 4300 ft [5]. In IFR conditions an arrival on one runway is independent of a departure on the other runway. Far parallel runways are spaced at least 4300 ft apart [5]. In IFR conditions the two runways can be operated independently for both arrivals and departures. Therefore, as noted earlier, the centerline separation of parallel runways determines the degree of interdependence between operations on each of the parallel runways. It should be recognized that in future the spacing requirements for simultaneous operations on parallel runways

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may be reduced. If this occurs, new spacing can be applied to the same classifications.

If the terminal buildings are placed between parallel runways, runways are always spaced far enough apart to allow room for the buildings, the adjoining apron, and the appropriate taxiways. When there are four parallel runways, each pair is spaced close, but the pairs are spaced far apart to provide space for terminal buildings.

In VFR conditions, close parallel runways allow simultaneous arrivals and departures, that is, arrivals may occur on one runway while departures are occurring on the other runway. Aircraft operating on the runways must have wingspans less than 171 ft (airplane design groups I through IV, see Table 6-2) for centerline spacing at the

minimum of 700 ft [5]. If larger wingspan aircraft are operating on these runways (airplane design groups V and VI), the centerline spacing
must be at least 1200 ft for such simultaneous operations [5]. In either case, wake vortex avoidance procedures must be used for simultaneous
operations on closely spaced parallel runways. Furthermore, simultaneous arrivals to both runways or simultaneous departures from both runways are not allowed in VFR conditions for closely spaced
parallel runways. In IFR conditions, closely spaced parallel runways cannot
be used simultaneously but may be operated as dual-lane runways.

Intermediate parallel runways may be operated with simultaneous arrivals in VFR conditions. Intermediate parallel runways may be operated in IFR conditions with simultaneous departures in a nonradar
environment if the centerline spacing is at least 3500 ft and in a radar environment if the centerline spacing is at least 2500 ft [5].

Simultaneous arrivals and departures are also permitted if the centerline

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spacing is at least 2500 ft if the thresholds of the runways are not staggered [5]. There are times when it may be desirable to stagger the thresholds of parallel runways. The staggering may be necessary because of the shape of the acreage available for runway construction, or it may be desirable for reducing the taxiing distance of takeoff and landing aircraft. The reduction in taxiing distance, however, is based on the premise that one runway is to be used exclusively for takeoff and the other for landing. In this case the terminal buildings are located between the runways so that the taxiing distance for each type of operation (takeoff or landing) is minimized. If the runway thresholds are staggered, adjustments to the centerline spacing requirement are allowed for simultaneous arrivals and departures [5]. If the arrivals are on the near threshold then the centerline spacing may be reduced by 100 ft for each 500 ft of threshold stagger down to a minimum centerline separation of 1000 ft for aircraft with wingspans up to 171 ft and a minimum of 1200 ft for larger wingspan aircraft. If the arrivals are on the far threshold the centerline spacing must be increased by 100 ft for each 500 ft of threshold stagger. Simultaneous arrivals in IFR conditions are not permitted on intermediate parallel runways but are permitted on far parallel runways with centerline spacings of at least 4300 ft [5].

The hourly capacity of a pair of parallel runways in VFR conditions varies greatly from 60 to 200 operations per hour depending on the aircraft mix and the manner in which arrivals and departures are processed on these runways [4]. Similarly, in IFR conditions the hourly capacity of a pair of closely spaced parallel runways ranges from 50 to 60 operations per hour, of a pair of intermediate parallel runways from 60 to 75 operations per hour, and for a pair of far parallel runways from 100 to 125 operations per hour [4].

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A dual-lane parallel runway consists of two closely spaced parallel runways with appropriate exit taxiways. Although both runways can be used for mixed operations subject to the conditions noted above, the desirable mode of operation is to dedicate the runway farthest
from the terminal building (outer) for arrivals and the runway closest to the terminal building (inner) for departures. It is estimated that a dual-lane runway can handle at least 70 percent more traffic than a single runway in VFR conditions and about 60 percent more traffic than a single runway in IFR conditions. It is recommended that the two runways be spaced not less than 1000 ft apart (1200 ft, where particularly larger wingspan aircraft are involved). This spacing also provides sufficient distance for an arrival to stop between the two runways. A parallel taxiway between the runways will provide for a nominal increase in capacity, but is not essential. The major benefit of a dual-lane runway is to provide an increase in IFR capacity with minimal acquisition of land [7, 14].

Intersecting Runways

Many airports have two or more runways in different directions crossing
each other. These are referred to as intersecting runways. Intersecting runways are necessary when relatively strong winds occur from more than one direction, resulting in excessive crosswinds when only one runway is provided. When the winds are strong, only one runway of a pair of intersecting runways can be used, reducing the capacity of the airfield substantially. If the winds are relatively light, both runways

can be used simultaneously. The capacity of two intersecting runways depends on the location of the intersection (i.e., midway or near the ends), the manner in which the runways are operated for takeoffs and landings, referred to as the runway use strategy, and the aircraft mix. The farther the intersection is from the takeoff end of the runway and the landing threshold, the lower is the capacity. The highest

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capacity is achieved when the intersection is close to the takeoff and landing threshold.

Open-V Runways

Runways in different directions which do not intersect are referred to as open-V runways. This configuration is shown in Fig. 6-4. Like intersecting runways, open-V runways revert to a single runway when winds are strong from one direction. When the winds are light, both runways may be used simultaneously.

The strategy which yields the highest capacity is when operations are away from the V and this is referred to as a diverging pattern. In VFR the hourly capacity for this strategy ranges from 60 to 180 operations per hour, and in IFR the corresponding capacity is from 50 to 80 operations per hour [4]. When operations are toward the V it is referred to as a converging pattern and the capacity is reduced to 50 to 100 operations per hour in VFR and to between 50 and 60 operations per hour in IFR [4].

Combinations of Runway Configurations

From the standpoint of capacity and air traffic control, a single-direction runway configuration is most desirable. All other things being equal, this configuration will yield the highest capacity compared with other configurations. For air traffic control the routing of aircraft in a single direction is less complex than routing in multiple directions. Comparing the divergent configurations, the open-V runway pattern is more desirable than an intersecting runway configuration. In the open -V configuration an operating strategy that routes aircraft away from the V will yield higher capacities than if the operations are reversed. If intersecting runways cannot be avoided, every effort should be made to place the intersections of both runways as close as possible to their thresholds and to operate the aircraft away from the intersection rather than toward the intersection.

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Figure 6-5 illustrates the complex runway configuration of Chicago‘s O‘Hare Field, with multiple parallel, intersecting, and nonintersecting

runways. It should be noted that a large capital improvement program is being undertaken to simplify the runway configuration, by adding additional parallel runways and removing many intersecting runways. This runway redesign is being done with the intention of improving the capacity and efficiency of airport operations

at the airport.

Runway Orientation

The orientation of a runway is defined by the direction, relative to magnetic north, of the operations performed by aircraft on the runway.

Typically, but not always, runways are oriented in such a manner that they may be used in either direction. It is less preferred to orient a runway in such a way that operating in one direction is precluded,

normally due to nearby obstacles.

In addition to obstacle clearance considerations, which will be discussed

later in this chapter, runways are typically oriented based on the area‘s wind conditions. As such, an analysis of wind is essential for

planning runways. As a general rule, the primary runway at an airport should be oriented as closely as practicable in the direction of the prevailing winds. When landing and taking off, aircraft are able to maneuver on a runway as long as the wind component at right angles to the direction of travel, the crosswind component, is not excessive. The FAA recommends that runways should be oriented so that aircraft may be landed at least 95 percent of the time with allowable crosswind components not exceeding specified limits based upon the airport reference code associated with the critical aircraft that has the shortest wingspan or slowest approach speed.

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When the wind coverage is less than 95 percent a crosswind runway is recommended.

The allowable crosswind is 10.5 kn (12 mi/h) for Airport Reference Codes A -I and B- I, 13 kn (15 mi/h) for Airport Reference Codes A-II and B-II, 16 kn (18.5 mi/h) for Airport Reference Codes A-III, B-III, C-I, C-II, C-III and C-IV, and 20 knots (23 mph) for Airport Reference

Codes A-IV through D-VI [5].

ICAO also specifies that runways should be oriented so that aircraft may be landed at least 95 percent of the time with crosswind components of 20 kn (23 mph) for runway lengths of 1500 m more, 13 kn (15 mi/h) for runway lengths between 1200 and 1500 m, and 10 kn (11.5 mi/h) for runway lengths less than 1200 m [1, 2].

Once the maximum permissible crosswind component is selected, the most desirable direction of runways for wind coverage can be determined by examination of the average wind characteristics at the airport under the following conditions:

1. The entire wind coverage regardless of visibility or cloud ceiling
2. Wind conditions when the ceiling is at least 1000 ft and the visibility is at least 3 mi
3. Wind conditions when ceiling is between 200 and 1000 ft and/or the visibility is between . and 3 mi.

The first condition represents the entire range of visibility, from excellent to very poor, and is termed the all weather condition. The next condition represents the range of good visibility conditions not requiring the use of instruments for landing, termed visual meteorological

condition (VMC). The last condition represents various degrees of poor visibility requiring the use of instruments for landing, termed instrument meteorological conditions (IMC).

The 95 percent criterion suggested by the FAA and ICAO is applicable

to all conditions of weather; nevertheless it is still useful to

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examine the data in parts whenever this is possible.

In the United States, weather records can be obtained from the Environmental Data and Information Service of the National Climatic Center at the National Oceanic and Atmospheric Administration located in Ashville, N.C., or from various locations found on the Internet.

Weather data are collected from weather stations throughout the United States on an hourly basis and recorded for analysis. The data collected include ceiling, visibility, wind speed, wind direction, storms, barometric pressure, the amount and type of liquid and frozen precipitation, temperature, and relative humidity. A report illustrating the tabulation and representation of some of the data of use in airport studies was prepared for the FAA [15]. The weather records contain the percentage of time certain combinations of ceiling and visibility occur (e.g., ceiling, 500 to 900 ft; visibility, 3 to 6 mi), and the

percentage of time winds of specified velocity ranges occur from different directions (e.g., from NNE, 4 to 7 mi/h). The directions are referenced to true north.

The Wind Rose

The appropriate orientation of the runway or runways at an airport can be determined through graphical vector analysis using a wind rose. A standard wind rose consists of a series of concentric circles cut

by radial lines using polar coordinate graph paper. The radial lines are drawn to the scale of the wind magnitude such that the area between each pair of successive lines is centered on the wind direction.

The shaded area indicates that the wind comes from the southeast (SE) with a magnitude between 20 and 25 mi/h. A template is also drawn to the same radial scale representing the crosswind component limits. A template drawn with crosswind component limits

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of 15 mi/h is shown on the right side of Fig. 6-7. On this template three equally spaced parallel lines have been plotted. The middle line represents the runway centerline, and the distance between the middle line and each outside line is, to scale, the allowable crosswind component (in this case, 15 mi/h). The template is placed over the wind rose in such a manner that the centerline on the template passes through the center of the wind rose.

By overlaying the template on the wind rose and rotating the centerline of the template through the origin of the wind rose one may determine the percentage of time a runway in the direction of the centerline of the template can be used such that the crosswind component

does not exceed 15 mi/h. Optimum runway directions can be determined from this wind rose by the use of the template, typically made on a transparent strip of material. With the center of the wind rose as a pivot point, the template is rotated until the sum of the percentages included between the outer lines is a maximum. If a wind vector from a segment lies outside either outer line on the template for the given direction of the runway, that wind vector must have a crosswind component which exceeds the allowable crosswind component plotted on the template. When one of the outer lines on the template divides a segment of wind direction, the fractional part is estimated visually to the nearest 0.1 percent. This procedure is consistent with the accuracy of the wind data and assumes that the wind percentage within the sector is uniformly distributed within that sector.

In practice, it is usually easier to add the percentages contained in the

sectors outside of the two outer parallel lines and subtract these from 100 percent to find the percentage of wind coverage.

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Estimating Runway Length

Other than orientation, planning and designing the length of a runway is critical to whether or not a particular aircraft can safely use the runway for takeoff or landing. Furthermore, designing a runway to accommodate a given aircraft is a difficult task, given the fact that an aircraft‘s required runway length will vary based on aircraft weight, as well as on several ambient conditions.

As a guide to airport planners, the FAA has published Advisory Circular 150/5325-4b, –Runway Length Requirements for Airport Design‖ [17]. In this publication, procedures are defined for estimating

the design runway length of aircraft, based on their maximum takeoff weights (MTOW), certain aircraft performance specifications, and the airport‘s field elevation and temperature. The airport

design runway length is found for the critical aircraft, defined as the aircraft which flies the greatest nonstop route segment from the airports

at least 500 operations per year and requires the longest runway.

The FAA‘s procedure for estimating runway length is based on the following data:

1. Designation of a critical aircraft
2. The maximum takeoff weight of the critical aircraft at the airport

3. The airport elevation
4. The mean daily maximum temperature for the hottest month at the airport
5. The maximum different in elevation along the runway centerline.

For the purposes of estimating runway length requirements, the

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Runway System Geometric Specifications

The runway system at an airport consists of the structural pavement, the shoulders, the blast pad, the runway safety area, various obstruction-free surfaces, and the runway protection zone,

1. The runway *structural pavement* supports the aircraft with respect to structural load, maneuverability, control, stability, and other operational and dimensional criteria.
2. The *shoulder* adjacent to the edges of the structural pavement resists jet blast erosion and accommodates maintenance and emergency equipment.
3. The *blast pad* is an area designed to prevent erosion of the surfaces adjacent to the ends of runways due to jet blast or propeller wash.
4. The *runway safety area* (RSA) is an area surrounding the runway prepared or suitable for reducing the risk of damage to aircraft in the event of an undershoot, overshoot, or excursion from the runway. ICAO refers to an area similar to the runway safety area as the *runway strip* and the *runway end safety area*. The runway safety area includes the structural pavement, shoulders, blast pad, and stopway, if provided. This area should be capable of supporting emergency and maintenance equipment as well as providing support for aircraft.
The runway safety area is cleared, drained, and graded and should have no potentially hazardous ruts, humps, depressions, or other surface variations. It should be free of objects except for objects that are required to be located in the runway safety area because of their function. These objects are required to be constructed on frangible mounted structures at the lowest possible height with the frangible point no higher than 3 in above grade.
5. The runway *object-free area* (OFA) is defined by the FAA as a two-dimensional ground area surrounding the runway which must be clear of parked aircraft and objects other than those

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whose location is fixed by function.

6. The runway *obstacle-free zone* (OFZ) is a defined volume of airspace centered above the runway which supports the transition between ground and airborne operations. The FAA specifies this as the airspace above a surface whose elevation is the same as that of the nearest point on the runway centerline and extending 200 ft beyond each end of the runway.

7. The *inner approach obstacle-free zone*, which applies only to runways with approach lighting systems, is the airspace above a surface centered on the extended runway centerline beginning 200 ft beyond the runway threshold at the same elevation as the runway threshold and extending 200 ft beyond the last light unit on the approach lighting system. Its width is the same as the runway obstacle-free zone and it slopes upward at the rate of 50 horizontal to 1 vertical.

8. The *inner transitional obstacle-free zone*, which applies only to precision instrument runways, is defined by the FAA as the volume of airspace along the sides of the runway and the inner approach obstacle-free zone. The surface slopes at the rate of 3 horizontal to 1 vertical out from the edge of the runway obstacle-free zone and the inner approach obstacle-free zone until it reaches a height of 150 ft above the established airport elevation.

9. The *runway protection zone* (RPZ) is an area on the ground used to enhance the protection of people and objects near the runway approach.

*Facilities for small airplanes only.

†From end of runway; with the declared distance concept, these lengths begin at the stop end of each ASDA and both ends of the LDA, whichever is greater.

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‡For runways serving small aircraft only; for large aircraft the greater of 400 ft or 180 ft plus the wingspan of the most demanding aircraft plus 20 ft for
each 1000 ft of airport elevation.
§For runways serving small aircraft with approach speeds of less than 50 kn; increase to 250 ft for runways serving aircraft with approach speeds greater
than 50 kn.
¶Beyond the end of each runway.
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Parallel Runway System Spacing

The spacing of parallel runways depends on a number of factors such as whether the operations are in VMC or IMC and, if in IMC, whether it is desired to have the capability of accommodating simultaneous arrivals or simultaneous arrivals and departures. At those airports serving both heavy and light aircraft simultaneous use of runways even in VMC conditions may be dictated by separation requirements to safeguard against wake vortices.

*a*For airplane design group III serving aircraft with maximum certified takeoff weight
greater than 150,000 lb, the standard runway width is 150 ft, the shoulder width is 25 ft,
and the blast pad width is 200 ft.
*b*Airplane design groups V and VI normally require stabilized or paved shoulder surfaces.
*c*For Airport Reference Code C-I and C-II, a runway safety area width of 400 ft is permissible.
For runways designed after 2/28/83 to serve aircraft approach category D aircraft,
the runway safety area width increases 20 ft for each 1000 ft of airport elevation
above mean sea level.

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*d*From end of runway; with the declared distance concept, these lengths begin at the stop
end of each ASDA and both ends of the LDA, whichever is greater. *e*For large aircraft the greater of 400 ft or 180 ft plus the wingspan of the most demanding
aircraft plus 20 ft for each 1000 ft of airport elevation; for small aircraft 300 ft for precision
instrument runways, 250 ft for all other runways serving small aircraft with
approach speeds of 50 kn or more, and 120 ft for all other runways serving small aircraft
with approach speeds less than 50
kn. *f*Beyond the end of each runway.

Under VMC, the FAA requires parallel runway centerline separations of 700 ft for all aircraft when the operations are in the same direction and wake vortices are not prevalent. It also recommends increasing the separation to 1200 ft for airplane design group V and VI runways. If
wake vortices are generated by heavy jets and it is desired to operate on
two runways simultaneously in VMC when little or no crosswind is present, the minimum distance specified by the FAA is 2500 ft.

For operations under VMC, the ICAO recommends that the minimum separations between the centerlines of parallel runways for simultaneous use disregarding wake vortices be 120 m (400 ft) for aerodrome code number 1, 150 m (500 ft) for aerodrome code number 2, and 210 m (700 ft) for aerodrome code number 3 or 4 runways. In IMC conditions, the FAA specifies 4300 ft and ICAO specifies 1525 m (5000 ft) as the minimum separation between centerlines of

*The width of a precision approach runway should not be less than 30 m where the aerodrome
code number is 1 or 2.

†Minimum width of pavement and shoulders when pavement width is less than 60 m.

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‡Symmetrical about the runway centerline.
§It is recommended that this be provided for the first 150 m from each end of the runway and
that it should be increased linearly from this point to a width of 210 m at a point 300 m from
each end of the runway and remain at this width for the remainder of the runway.

parallel runways for simultaneous instrument approaches. However, there is evidence that these distances are conservative and steps are being taken to reduce it. The ultimate goal is to reduce this distance by about one-half. For dependent instrument approaches both the FAA and ICAO recommend centerline separations of 3000 ft (915 m). For triple and quadruple simultaneous instrument approaches, the FAA requires 5000-ft separation between runway centerlines, although will allow 4300 ft separations on a case-by-case basis. Both the FAA and ICAO specify that two parallel runways may
be used simultaneously for radar departures in IMC if the centerlines are separated by at least 2500 ft (760 m). The FAA requires a 3500-ft centerline separation for simultaneous nonradar departures. If two parallel runways are to be operated independently of each other in IMC under radar control, one for arrivals and the other for departures, both the FAA and ICAO specify that the minimum separation between the centerlines is 2500 ft (760 m) when the thresholds are even. If the thresholds are staggered, the runways can be brought closer together or must be separated farther depending on the amount of the stagger and which runways are used for arrivals and departures. If approaches are to the nearest runway, then the spacing may
be reduced by 100 ft (30 m) for each 500 ft (150 m) of stagger down to
a minimum of 1200 ft (360 m) for airplane design groups V and VI and 1000 ft (300 m) for all other aircraft. However, if the approaches are to the farthest runway, then the runway spacing must be increased by 100 ft (30 m) for each 500 ft (150 m) of stagger.

Sight Distance and Longitudinal Profile

The FAA requirement for sight distance on individual runways requires that the runway profile permit any two points 5 ft above the runway centerline to be mutually visible for the entire runway length. If, however, the runway has a full length parallel taxiway, the runway profile may be such that an unobstructed line of sight will exist from any point 5 ft above the runway centerline to any other point 5 ft above the runway centerline for one-half the runway length.

The FAA recommends a clear line of sight between the ends of intersecting runways. The terrain must be graded and permanent objects designed and sited so that there will be an unobstructed line of sight from any point 5 ft above one runway centerline to any point 5 ft above an intersecting runway centerline within the runway visibility zone. The runway visibility zone is the area formed by imaginary lines connecting the visibility points of the two intersecting runways. The runway visibility zone for intersecting runways is shown in Fig. 6-21. The visibility points are defined as follows:

1. If the distance from the intersection of the two runway centerlines is 750 ft or less, the visibility point is on the centerline at the runway end designated by point a in Fig. 6-21.
2. If the distance from the intersection of the two runway centerlines is greater than 750 ft but less than 1500 ft, the visibility point is on the centerline 750 ft from the intersection of the centerlines designated by point b in Fig. 6-21.
3. If the distance from the intersection of the two runway centerlines is equal to or greater than 1500 ft, the visibility point is on the centerline equidistant from the runway end and the intersection of the centerlines designated by points c and d in Fig. 6-21.

The ICAO requirement for sight distance on individual runways requires that the runway profile permit an unobstructed view between any two points at a specified height above the runway centerline

to be mutually visible for a distance equal to at least one-half

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the runway length. ICAO specifies that the height of these two points be 1.5 m (5 ft) above the runway for aerodrome code letter A runways,
2 m (7 ft) above the runway for aerodrome code letter B runways, and 3 m (10 ft) above the runway for aerodrome code letter C, D, or E runways.
It is desirable to minimize longitudinal grade changes as much as possible. However, it is recognized that this may not be possible for reasons of economy. Therefore both the ICAO and FAA allow changes

*a*Applies also to runway safety area adjacent to sides of the runway. *b*May not exceed 0.8 percent in the first and last quarter of runway. *c*A minimum of 3 percent for turf.
*d*A slope of 5 percent is recommended for a 10 ft width adjacent to the pavement areas to promote drainage.
*e*For the first 200 ft from the end of the runway and if it slopes it must be downward. For the remainder of the runway safety area the slope must be such that any upward slope does not penetrate the approach surface or clearway plane and any downward slope does not exceed 5 percent.
*f*For each 1 percent change in grade.
*g*No vertical curve is required if the grade change is less than 0.4 percent.
*h*Distance is multiplied by the sum of the absolute grade grade changes in percent.
Source: Federal Aviation Administration [6].
longitudinal gradient and longitudinal grade changes to 2 percent for runways serving approach category A and B aircraft and 1.5 percent for runways serving approach category C, D, and E aircraft. ICAO limits both longitudinal gradient and longitudinal grade changes to

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2 percent for aerodrome code number 1 and 2 runways and 1.5 percent
for aerodrome code number 3 runways. For aerodrome code number 4 runways the maximum longitudinal gradient is 1.25 percent and the maximum change in longitudinal gradient is 1.5 percent. In addition, for runways that are equipped to be used in bad weather, the gradient of the first and last quarter of the length of the runway must be very flat for reasons of safety. Both the ICAO and the FAA require that this
gradient not exceed 0.8 percent. In all cases it is desirable to keep both
longitudinal grades and grade changes to a minimum. Longitudinal slope changes are accomplished by means of vertical
curves. The length of a vertical curve is determined by the magnitude of
the changes in slope and the maximum allowable change in the slope of
the runway.

*May not exceed 0.8 percent in the first and last quarter of runway for aerodrome code
number 4 or for a category II or III precision instrument runway for aerodrome code
number 3.

†Difference in elevation between high and low point divided by runway length

‡For each 1 percent change in grade.

§Distance is multiplied by sum of absolute grade changes in percent minimum length is 45 m.

The number of slope changes along the runway is also limited.
The FAA requires that the distance between the points of intersection of two successive curves should not be less than the sum of the absolute
percentage values of change in slope multiplied by the 250 ft for

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airports serving aircraft approach category A and B aircraft and 1000 ft
for airports serving aircraft approach category C, D, and E aircraft.
The ICAO requires that the distance between the points of intersection
of two successive curves should not be less than the sum of the absolute percentage values of change in slope multiplied by 50 m (165 ft) for aerodrome code number 1 and 2 runways, 150 m (500 ft) for aerodrome code number 3 runways, and 300 m (1000 ft) for aerodrome
code number 4 runways. ICAO also specifies that the minimum distance in all cases is 45 m (150 ft).
For example, for an FAA runway serving transport aircraft, that is, approach category C, D, or E aircraft, if the change in slope was 1.5 percent, the required length of vertical curve would be 1500 ft. Vertical
curves are normally not necessary if the change in slope is not more than 0.4 percent. The FAA specifies a minimum length of vertical transition
curve of 300 for each 1 percent change in grade for runways

*For facilities for small aircraft only.
†Satisfies the requirement that no part of an aircraft at a holding an increase to these
separations may be needed to achieve this result.
‡For sea level up to elevation 6000 ft. Increase by 1 ft for each 100 ft of airport elevation
above 6000 ft.
serving approach category A and B aircraft and 1000 ft for each 1 percent
change in grade for airport serving approach category C, D, and E aircraft. ICAO specifies a minimum length of vertical transition curve of 75 m for each 1 percent change in grade for aerodrome code number

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1 runways, 150 m for each 1 percent change in grade aerodrome code number 2 runways, and 300 m for each 1 percent change in grade for aerodrome code number 4 runways.

Transverse Gradient

A typical cross section of a runway is shown in Fig. 6-23. The FAA and ICAO specifications for transverse slope on the runways are given in Tables 6-10 and 6-11, respectively. It is recommended that a 5 percent transverse slope be provided for the first 10 ft of shoulder adjacent to a pavement edge to ensure proper drainage.

*a*Lighting of obstacle penetrations to this surface or the use of a VGSI, as defined by the TERPS order, may avoid displacing the threshold.

*b*10,000 ft is a nominal value for planning purposes. The actual length of these areas is dependent upon the visual descent point position for 20:1 and
34:1 and decision altitude point for the 30:1.

*c*Any penetration to this surface will limit the runway end to nonprecision approaches. No vertical approaches will be authorized until the penetration(s)
is/are removed except obstacles fixed by function and/or allowable grading.

*d*Dimension A is measured relative to departure end of runway (DER) or TODA (to include clearway).

*e*Data collected regarding penetrations to this surface are provided for information and use by the air carriers operating from the airport. These requirements
do not take effect until January 1, 2009.

*f*Surface dimensions/obstacle clearance surface (OCS) slope represent a nominal approach with 3° GPA, 50′ TCH, 500′ HAT. For specific cases refer to

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TERPS. The obstacle clearance surface slope (30:1) represents a nominal approach of 3° (also known as the glide path angle). This assumes a threshold crossing height of 50 ft. Three degrees is commonly used for ILS systems and VGSI aiming angles. This approximates a 30:1 approach angle that is between the 34:1 and the 20:1 notice surfaces of Part 77. Surfaces cleared to 34:1 should accommodate a 30:1 approach without any obstacle clearance problems.

*g*For runways with vertically guided approaches the criteria in Row 7 is in addition to the basic criteria established within the table, to ensure the protection of the glide path qualification surface.

*h*For planning purposes, sponsors and consultants determine a tentative decision altitude based on a 3° glide path angle and a 50-ft threshold crossing height.

These specifications are used to site the location of a runway‘s threshold so that approach and departure procedures associated with that runway are not adversely affected by existing obstacles or terrain. The siting specifications vary depending on a number of runway use conditions, including

• The approach speed of arriving aircraft
• The approach category of arriving aircraft
• Day versus night operations
• Types of instrument approaches
• The presence of published instrument departure procedures
• The use of the runway by air carriers

Runway end siting requirements are often the most confusing as well as overlooked element of runway planning. Care should be given to fully understand the purpose of the planned runway, the type of aircraft that will be using the runway, the current and future

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instrument approach procedures associated with the runway, and of course any terrain or obstacles in the vicinity.
Should an object penetrate any of the surfaces at the site of a runway, Displacing the threshold allows the airport planner to design runways with sufficient
lengths to accommodate aircraft departures, while also allowing arrivals to safely approach the runway by maintaining sufficient clearance from upstream obstacles. Displacing the threshold does carry the penalty of reducing available runway lengths for landing. The FAA recommends avoiding the need for displaced thresholds when possible, but recognizes their benefits in the wake of no other alternatives.

Taxiways and Taxilanes

Taxiways are defined paths on the airfield surface which are established
for the taxiing of aircraft and are intended to provide a linkage between one part of the airfield and another. The term ―dual parallel taxiways‖ refers to two taxiways parallel to each other on which airplanes
can taxi in opposite directions. An apron taxiway is a taxiway located usually on the periphery of an apron intended to provide a through taxi route across the apron. A taxilane is a portion of the aircraft

parking area used for access between the taxiways and the aircraft parking positions. ICAO defines an aircraft stand taxilane as a portion of the apron intended to provide access to the aircraft stands only. In order to provide a margin of safety in the airport operating areas, the trafficways must be separated sufficiently from each other and from adjacent obstructions. Minimum separations between the centerlines of taxiways, between the centerlines of taxiways and taxilanes,
and between taxiways and taxilanes and objects are specified in order that aircraft may safely maneuver on the airfield.

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Widths and Slopes

Since the speeds of aircraft on taxiways are considerably less than on runways, criteria governing longitudinal slopes, vertical curves, and sight distance are not as stringent as for runways. Also the lower speeds permit the width of the taxiway to be less than that of the runway.

*a*For airplanes in airplane design group III with a wheelbase equal to or greater than 60 ft,
the standard taxiway width is 60 ft.
*b*The taxiway edge safety margin is the minimum acceptable between the outside of the
airplane wheels and the pavement edge.
*c*For airplanes in airplane design group III with a wheelbase equal or greater than 60 ft,
the taxiway edge safety margin is 15 ft.
*d*Airplanes in airplane design groups V and VI normally stabilized or paved taxiway
shoulder surfaces.
*e*May use aircraft wingspan in lieu of these values. *f*May
use 1.4 wingspan plus 20 ft in lieu of these values. *g*May
use 1.2 wingspan plus 20 ft in lieu of these values. *h*May
use 1.2 wingspan plus 10 ft in lieu of these values. *i*May
use 0.7 wingspan plus 10 ft in lieu of these values. *j*May
use 1.1 wingspan plus 10 ft in lieu of these values. *k*May
use 0.6 wingspan plus 10 ft in lieu of these values.

Taxiway and Taxilane Separation Requirements FAA Separation Criteria

The separation criteria adopted by the FAA are predicated upon the wingtips of the aircraft for which the taxiway and taxilane system have been designed and provide a minimum wingtip clearance on these facilities. The required separation between taxiways, between a taxiway and a taxilane, or between a taxiway and a fixed or movable

object requires a minimum wingtip clearance of 0.2 times the wingspan
of the most demanding aircraft in the airplane design group
plus 10 ft. This clearance provides a minimum taxiway centerline to a parallel taxiway centerline or taxilane centerline separation of 1.2 times the wingspan of the most demanding aircraft plus 10 ft, and between a taxiway centerline and a fixed or movable object of 0.7 times the wingspan of the most demanding aircraft plus 10 ft. This

*A minimum of 3 percent for turf.
†A slope of 5 percent is recommended for a 10-ft width adjacent to the pavement areas to
promote drainage.
‡For each 1 percent of grade change.
§Distance is multiplied by the sum of the absolute grade changes in percent.

*18 m if used by aircraft with a wheelbase equal to or greater than 18 m.
†23 m is used by aircraft with an outer main gear wheel span equal to or greater than 9 m.
‡4.5 m. if intended to be used by airplane with a wheelbase equal to or greater than 18 m.
separation is also applicable to aircraft traversing through a taxiway on an apron or ramp. This separation may have to be increased to accommodate pavement widening on taxiway curves. It is recommended
that a separation of at least 2.6 times the wheelbase of the
most demanding aircraft be provided to accommodate a 180 turn when the pavement width is designed for tracking the nose wheel on the centerline.
The taxilane centerline to a parallel taxilane centerline or fixed or movable object separation in the terminal area is predicated on a wingtip clearance of approximately half of that required for an apron

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taxiway. This reduction in clearance is based on the consideration that taxiing speed is low in this area, taxiing is precise, and special guidance techniques and devices are provided. This requires a wingtip clearance or wingtip-to-object clearance of 0.1 times the wingspan of the most demanding aircraft plus 10 ft. Therefore, this establishes a minimum separation between the taxilane centerlines of 1.1 times the wingspan of the most demanding aircraft plus 10 ft, and between a taxilane centerline and a fixed or movable object of 0.6 times the wingspan of the most demanding aircraft plus 10 ft [6]. Therefore, when dual parallel taxilanes are provided in the terminal apron area, the taxilane object- free area becomes 2.3 the wingspan of the most demanding aircraft plus 30 ft.

The separation criteria adopted by ICAO are also predicated upon the wingtips of the aircraft for which the taxiway and taxilane system have been designed and providing a minimum wingtip clearance on these facilities, but also consider a minimum clearance between the outer main gear wheel and the taxiway edge. The required separation between taxiways or between a taxiway and a taxilane requires a minimum wingtip clearance, $C1$, of 3 m for aerodrome code letter A and B runways, 4.5 m for aerodrome code letter C runways, and

*For each 1 percent of grade change.

7.5 m for aerodrome code letter D and E runways. The minimum clearance between the edge of each taxiway and the outer main gear wheels, the taxiway edge safety margin $U1$, is given in Table 6-20. This clearance provides a minimum taxiway centerline to a parallel taxiway centerline or taxilane centerline separation given by Eq. (6-1).

STT WS 2U1 C1 (6-1)

where STT minimum taxiway-to-taxiway or taxiway-to-taxilane separation

WS wingspan of the most demanding aircraft U1

taxiway edge safety margin

*C*1 minimum wingtip clearance
Therefore, for example, an ICAO aerodrome code letter E runway, which accommodates aircraft with wingspans up to 65 m, requires a taxiway centerline to a taxiway centerline or a taxilane centerline separation from Eq. (6-1) of 65 2(4.5) 7.5 81.5 m. The required separation between a taxiway centerline or an apron taxiway centerline and a fixed or movable object is found from Eq. (6-2).

*S*TO 0.5 WS *U*1 *C*2 (6-2)

where *S*TO is the minimum taxiway or apron taxiway to a fixed or movable object separation and *C*2 is the required clearance between a wingtip and an object.

The required clearance between a wingtip and an object *C*2 is 4.5 m for aerodrome code letter A runways, 5.25 m for aerodrome code letter B runways, 7.5 m for aerodrome code letter C runways, and 12 m for aerodrome code letter D and E runways.

The required separation between an aircraft stand taxilane centerline and a fixed or movable object is found from Eq. (6-3).

*S*ATO 0.5 WS *U*2 *C*1 (6-3)

where *S*ATO is the minimum aircraft stand taxilane to fixed or movable object separation and *U*2 is the aircraft stand safety margin.

Since aircraft moving on the aircraft stand taxilane are moving at low speed and are often under positive ground guidance, the aircraft stand safety margin is less than on the taxiway system. The value for this safety margin *U*2 is 1.5 m for aerodrome code letter A and B airports, 2 m for aerodrome code letter C airports, and 2.5 m for aerodrome code letter D or E airports. The taxiway and taxilane separation criteria adopted by ICAO are given in Table 6-20.

Sight Distance and Longitudinal Profile

As in the case of runways, the number of changes in longitudinal

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profile for taxiways is limited by sight distance and minimum distance

between vertical curves.
The FAA does not specify line of sight requirements for taxiways other than those discussed earlier related to runway and taxiway intersections. However, the sight distance along a runway from an intersecting taxiway needs to be sufficient to allow a taxiing aircraft to
enter or cross the runway safely. The FAA specifies that from any point
on the taxiway centerline the difference in elevation between that point and the corresponding point on a parallel runway, taxiway, or apron edge is 1.5 percent of the shortest distance between the points.
ICAO requires that the surface of the taxiway should be seen for a distance of 150 m from a point 1.5 m above the taxiway for aerodrome
code letter A runways, for a distance of 200 m from a point 2 m above the taxiway for aerodrome code letter B runways, and for a distance of 300 m from a point 3 m above the taxiway for aerodrome code letter C, D, or E runways.
In regard to longitudinal profile of taxiways, the ICAO does not specify the minimum distance between the points of intersection of vertical curves. The FAA specifies that the minimum distance for both
utility and transport category airports should be not less than the product of 100 ft multiplied by the sum of the absolute percentage values of change in slope.

Exit Taxiway Geometry

The function of exit taxiways, or runway turnoffs as they are sometimes
called, is to minimize runway occupancy by landing aircraft. Exit taxiways can be placed at right angles to the runway or some other angle to the runway. When the angle is on the order of 30°, the term high-speed exit is often used to denote that it is designed for

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higher speeds than other exit taxiway configurations. In this chapter, specific dimensions for high -speed exit, right-angle exit (low-speed) taxiways are presented. The dimensions presented here are the results obtained from research conducted many years ago [13] and subsequent research conducted by the FAA.

The earlier tests [13] were conducted on wet and dry concrete and asphalt pavement with various types of civil and military aircraft in order to determine the proper relationship between exit speed and radii of curvature and the general configuration of the taxiway. A significant finding of the tests was that at high speeds a compound curve was necessary to minimize tire wear on the nose gear and, therefore, the central or main curve radius *R*2 should be preceded by a much larger radius curve *R*1.

Aircraft paths in the test approximated a spiral. A compound curve is relatively easy to establish in the field and begins to approach the shape of a spiral, thus the reason for suggesting a compound curve. The following pertinent conclusions were reached as a result of the tests [13]:

1. Transport category and military aircraft can safely and comfortably turn off runways at speeds on the order of 60 to 65 mi/h on wet and dry pavements.
2. The most significant factor affecting the turning radius is speed, not the total angle of turn or passenger comfort.
3. Passenger comfort was not critical in any of the turning movements.
4. The computed lateral forces developed in the tests were substantially below the maximum lateral forces for which the landing gear was designed.
5. Insofar as the shape of the taxiway is concerned, a slightly widened entrance gradually tapering to the normal width of taxiway is preferred. The widened entrance gives the pilot

more latitude in using the exit taxiway.
6. Total angles of turn of 30° to 45° can be negotiated satisfactorily. The smaller angle seems to be preferable because the length of the curved path is reduced, sight distance is improved, and less concentration is required on the part of the pilots.
7. The relation of turning radius versus speed expressed by the formula below will yield a smooth, comfortable turn on a wet or dry pavement when *f* is made equal to 0.13.
8. The curve expressed by the equation for *R*2 should be preceded by a larger radius curve *R*1 at exit speeds of 50 to 60 mi/h. The larger radius curve is necessary to provide a gradual transition from a straight tangent direction section to a curved path section. If the transition curve is not provided tire wear on large jet transports can be excessive.
9. Sufficient distance must be provided to comfortably decelerate an aircraft after it leaves the runway. It is suggested that for the present this distance be based on an average rate of deceleration of 3.3 ft/s2. This applies only to transport category aircraft. Until more experience is gained with this type of operation the stopping distance should be measured from the edge of the runway.
A chart showing the relationship of exit speed to radii *R*1 and *R*2, and length of transition curve *L*1
ICAO has indicated the relationship between aircraft speed and the radius of curvature of taxiway curves
For high-speed exit taxiways ICAO recommends a minimum radius of curvature for the taxiway centerline of 275 m (900 ft) for aerodrome
code number 1 and 2 runways and 550 m (1800 ft) for aerodrome code
number 3 and 4 runways. This will allow exit speeds under wet conditions
of 65 km/h (40 mi/h) for aerodrome code number 1 and 2

runways and 93 km/h (60 mi/h) for aerodrome code number 3 and 4 runways. It also recommends a straight tangent section after the turnoff
curve to allow exiting aircraft to come to a full stop clear of the intersecting taxiway when the intersection is 30°. This tangent distance
should be 35 m (115 ft) for aerodrome code number 1 and 2 runways and 75 m (250 ft) for aerodrome code number 3 and 4 runways

[2, 4].
A configuration for an exit speed of 60 mi/h and a turnoff angle of 30° is shown in Fig. 6 -34. The FAA recommends that the taxiway centerline circular curve be preceded by a 1400-ft spiral to smooth the transition from the runway centerline to the taxiway exit circular curve. ICAO recommends the same geometry for both of these highspeed
exits. Right-angle or 90° exit taxiways, although not desirable from the standpoint of minimizing runway occupancy, are often

constructed for other reasons. The configurations for a 90° exit and other common taxiway intersection configurations are illustrated in Fig. 6-35. The dimensions labeled in Fig. 6-35 are determined by the aircraft design group of the design aircraft. These dimensional standards
are provided in Table 6-23.

Location of Exit Taxiways

The location of exit taxiways depends on the mix of aircraft, the approach
and touchdown speeds, the point of touchdown, the exit speed, the rate
of deceleration, which in turn depends on the condition of the pavement
surface, that is, dry or wet, and the number of exits.

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While the rules for flying transport aircraft are relatively precise, a certain amount of variability among pilots is bound to occur especially in respect to braking force applied on the runway and the distance from runway threshold to touchdown. The rapidity and the manner in which air traffic control can process arrivals is an extremely important factor in establishing the location of exit taxiways. The location of exit taxiways is also influenced by the location of the runways relative to the terminal area.

Several mathematical analyses or models have been developed for optimizing exit locations. While these analyses have been useful in providing an understanding of the significant parameters affecting location, their usefulness to planners has been limited because of the complexity of the analyses and a lack of knowledge of the inputs required for the application of the models. As a result greater use is made of much more simplified methods.

The landing process can be described as follows. The aircraft crosses the runway threshold and decelerates in the air until the main landing gear touches the surface of the pavement. At this point the nose gear has not made contact with the runway. It may take as long as 3 s to do so. When it does, reverse thrust or wheel brakes or a combination of both are used to reduce the forward speed of the aircraft to exit velocity. Empirical analysis has revealed that the average deceleration of air-carrier aircraft on the runway is about 5 ft/s2.

In the simplified procedure, an aircraft is assumed to touch down at 1.3 times the stall speed for a landing weight corresponding to 85 percent of the maximum structural landing weight. In lieu of computing the distance from threshold to touchdown, touchdown distances are assumed as fixed values for certain classes of aircraft. Typically these values range from 500 to 1500 ft from the runway

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threshold. To these distances are added the distances to decelerate to exit speed.

These locations are derived using standard sea-level conditions. Altitude and temperature can affect the location of exit taxiways. Altitude increases distance on the order of 3 percent for each 1000 ft above sea level and temperature increases the distance 1.5 percent for each 10°F above 59°F.

During runway capacity studies conducted for the FAA, data were collected on exit utilization at various large airports in the United States [18]. T indicate the

cumulative percentage of each class of aircraft which have exited the runway at exits located at various distances from the arrival threshold. It is recommended that the point of intersection of the centerlines of taxiway exits and runways, which are up to 7000 ft in length and accommodate aircraft approach category C, D, and E aircraft, should

be located about 3000 ft from the arrival threshold and 2000 ft from the stop end of the runway. To accommodate the average mix of aircraft on runways longer than 7000 ft, intermediate exits should be located at intervals of about 1500 ft. At airports where there are extensive operations with aircraft approach category A and B aircraft, an exit located between 1500 and 2000 ft from the landing threshold is recommended.

Planners often find that the runway configuration and the location of the terminal at the airport often preclude placing the exits at locations based on the foregoing analysis. This is nothing to be alarmed about since it is far better to achieve good utilization of the exits than to be too concerned about a few seconds lost in occupancy time.

When locating exits it is important to recognize local conditions such as frequency of wet pavement or gusty winds. It is far better to place the exits several hundred feet farther from the threshold than

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to have aircraft overshoot the exits a large amount of time. The standard deviation in time required to reach exit speed is on the order of 2 or 3 s. Therefore, if the exits were placed down the runway as much as two standard deviations from the mean, the loss in occupancy time would only be 4 to 6 s. In planning exit locations at specific airports, one needs to consult with the airlines relative to the specific performance characteristics of the aircraft intended for use at the airport.

The total occupancy time of an aircraft can be roughly estimated using the following procedure. The runway is divided into four components, namely, flight from threshold to touchdown of main gear, time required for nose gear to make contact with the pavement after the main gear has made contact, time required to reach exit velocity from the time the nose gear has made contact with the pavement and brakes have been applied, and time required for the aircraft to turn Mix Index***** Exit Range from Arrival Threshold

0–20 2000–4000 21–50 3000–5500 51–80 3500–6500 81–120 5000–7000

∗121–180 5500–7500

Mix Index is equal to the percentage of Class C aircraft plus three an aircraft with a maximum certified takeoff weight in excess of class D aircraft, where a class C aircraft is an aircraft with a maximum certified takeoff weight greater than 12,500 lb and up to 300,000 lb and a class D aircraft is an aircraft with a maximum certified takeoff weight in excess of 300,000 lb.

off on to the taxiway and clear the runway. For the first component it

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can be assumed that the touchdown speed is 5 to 8 kn less than the speed over the threshold. The rate of deceleration in the air is about 2.5 ft/s2. The second component is about 3 s and the third component depends upon exit speed. Time to turnoff from the runway will be on the order of 10 s. As may be observed in this table, typical runway occupancy times for 60 mi/h high-speed exits are 35 to 45 s. The corresponding time for a 15 mi/h regular exit is 45 to 60 s for air carrier aircraft.

Airport Lighting,Marking, and
Signage

Introduction

Visual aids assist the pilot on approach to an airport, as well as navigating
around an airfield and are essential elements of airport infrastructure. As such, these facilities require proper planning and precise design.

These facilities may be divided into three categories: lighting,

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marking, and signage. Lighting is further categorized as either approach
lighting or surface lighting. Specific lighting systems described in this chapter include
1. Approach lighting
2. Runway threshold lighting
3. Runway edge lighting
4. Runway centerline and touchdown zone lights
5. Runway approach slope indicators
6. Taxiway edge and centerline lighting

The proper placement of these systems is described in this chapter but no attempt has been made to describe in detail the hardware or its installation. Airfield marking and signage includes
1. Runway and taxiway pavement markings
2. Runway and taxiway guidance sign systems

Airfield lighting, marking, and signage facilities provide the following
functions:
1. Ground to air visual information required during landing
2. The visual requirements for takeoff and landing
3. The visual guidance for taxiing

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The Requirements for Visual Aids

Since the earliest days of flying, pilots have used ground references for
navigation when approaching an airport, just as officers on ships at sea have used landmarks on shore when approaching a harbor. Pilots need visual aids in good weather as well as in bad weather and during the day as well as at night.

In the daytime there is adequate light from the sun, so artificial lighting is not usually required but it is necessary to have adequate contrast in the field of view and to have a suitable pattern of brightness
so that the important features of the airport can be identified and

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oriented with respect to the position of the aircraft in space. These requirements are almost automatically met during the day when the weather is clear.
The runway for conventional aircraft always appears as a long narrow strip with straight sides and is free of obstacles. It can therefore be easily identified from a distance or by flying over the field. Therefore, the perspective view of the runway and other identifying reference landmarks are used by pilots as visual aids for orientation when they are approaching the airport to land. Experience has demonstrated that the horizon, the runway edges, the runway threshold, and the centerline
of the runway are the most important elements for pilots to see.
In order to enhance the visual information during the day, the runway is painted with standard marking patterns. The key elements in these patterns are the threshold, the centerline, the edges, plus multiple
parallel lines to increase the perspective and to define the plane of the surface.
During the day when visibility is poor and at night, the visual information is reduced by a significant amount over the clear weather daytime scene. It is therefore essential to provide visual aids which will be as meaningful to pilots as possible.

The Airport Beacon

Beacons are lighted to mark an airport. They are designed to produce a narrow horizontal and vertical beam of high-intensity light which is rotated about a vertical axis so as to produce approximately 12 flashes per minute for civil airports and 18 flashes per minute for military airports [28]. The flashes with a clearly visible duration of at least 0.15 s
are arranged in a white-green sequence for land airports and a whiteyellow
sequence for landing areas on water. Military airports use a double white flash followed by a longer green or yellow flash to differentiate

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them from civil airfields. The beacons are mounted on top of the control tower or similar high structure in the immediate vicinity of the airport.

Obstruction Lighting

Obstructions are identified by fixed, flashing, or rotating red lights or beacons. All structures that constitute a hazard to aircraft in flight or during landing or takeoff are marked by obstruction lights having a horizontally uniform intensity duration and a vertical distribution design to give maximum range at the lower angles (1.5° to 8°) from which a colliding approach would most likely come.

The Aircraft Landing Operation

An aircraft approaching a runway in a landing operation may be visualized as a sequence of operations involving a transient body suspended in a three-dimensional grid that is approaching a fixed two-dimensional grid. While in the air, the aircraft can be considered as a point mass in a three-dimensional orthogonal coordinate system in which it may have translation along three coordinate directions and rotation about three axes. If the three coordinate axes are aligned horizontal, vertical, and parallel to the end of the runway, the directions of motion can be described as lateral, vertical, and forward. The rotations are normally called pitch, yaw, and roll, for the horizontal, vertical, and parallel axes, respectively. During a landing operation, pilots must control and coordinate all six degrees of freedom of the aircraft so as to bring the aircraft into coincidence with the desired approach or reference path to the touchdown point on the runway. In order to do this, pilots need translation information regarding the aircraft's alignment, height, and distance, rotation information regarding pitch, yaw, and roll, and information concerning the rate of descent and the rate of closure with the desired path.

Alignment Guidance

Pilots must know where their aircraft is with respect to lateral displacement from the centerline of the runway. Most runways are from

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75 to 200 ft wide and from 3000 to 12,000 ft long. Thus any runway is a long narrow ribbon when first seen from several thousand feet above. The predominant alignment guidance comes from longitudinal lines that constitute the centerline and edges of the runway. All techniques, such as painting, lighting, or surface treatment that develop contrast and emphasize these linear elements are helpful in providing alignment information.

Height Information

The estimation of the height above ground from visual cues is one of the most difficult judgments for pilots. It is simply not possible to provide good height information from an approach lighting system. Consequently the best source of height information is the instrumentation in the aircraft. However, use of these instruments often requires the availability of precision ground or satellite based navigation technologies.

Many airports have no such technologies, and at others only provide lateral approach guidance to certain runways. Consequently two types of ground- based visual aids defining the desired glide path have been developed. These are known as the visual approach slope indicator (VASI) and the precision approach path indicator (PAPI) which are discussed later in this chapter.

Several parameters influence how much a pilot can see on the ground. One of these is the *cockpit cutoff angle*. This is the angle between the longitudinal axis of the fuselage and an inclined plane below which the view of the pilot is blocked by some part of the aircraft,

Approach Lighting

Approach lighting systems (ALS) are designed specifically to provide guidance for aircraft approaching a particular runway under nighttime or other low-visibility conditions. While under nighttime conditions

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it may be possible to view approach lighting systems from several miles away, under other low-visibility conditions, such as fog, even the most intense ALS systems may only be visible from as little as 2500 ft from the runway threshold.

Studies of the visibility in fog [3] have shown that for a visual range of 2000 to 2500 ft it would be desirable to have as much as 200,000 candelas (cd) available in the outermost approach lights where the slant range is relatively long. Under these same conditions the optimum intensity of the approach lights near the threshold should be on the order of 100 to 500 cd. A transition in the intensity of the light that is directed toward the pilot is highly desirable in order to provide the best visibility at the greatest possible range and to avoid glare and the loss of contrast sensitivity and visual acuity at short range.

System Configurations

The configurations which have been adopted are the Calvert system [3] shown in Fig. 8-3 which has been widely used in Europe and other parts of the world, the ICAO category II and category III system shown in Fig. 8-4, and the four system configurations which have been adopted by the FAA in the United States shown in Fig. 8-5. The FAA publishes criteria for the establishment of the approach lighting systems [13] and other navigation facilities at airports [6]. Approach lights are normally mounted on frangible pedestals of varying height to improve the perspective of the pilot in approaching a runway.

The first approach lighting system was known as the Calvert system. In this system, developed by E. S. Calvert in Great Britain in 1949, includes a line of single bulb lights spaced on 100-ft centers along the extended runway centerline and six transverse crossbars of lights of variable length spaced on 500-ft centers, for a total length of 3000 ft.

For operations in very poor visibility, ICAO has certified a modification of the Calvert system, known as the ICAO category II system. The variation calls for a higher lighting intensity to the inner 300 m of the system closest to the runway threshold. The category II and category III system adopted by ICAO consists of two lines of red bars on each side of the runway centerline and a single line of white bars on the runway centerline both at 30 m intervals and both extending out 300 m from the runway threshold. In addition, there are two longer bars of white light at a distance of 150 and 300 m from the runway threshold, and a long threshold bar of green light at the runway threshold. ICAO also recommends that the longer bars of white light also be placed at distances of 450, 500, and 750 m from the runway threshold if the runway centerline lights extend out that distance The ALSs currently certified by the FAA for installation in the United States consist of a high-intensity ALS with sequenced flashing lights (ALSF-2), which is required for category II and category III precision approaches, a high-intensity approach lighting system with sequenced flashing lights (ALSF-1), and three medium-intensity ALSs (MALSR, MALS, MALSF).

In each of these systems there is a long transverse crossbar located 1000 ft from the runway threshold to indicate the distance from the runway threshold. In these systems roll guidance is provided by crossbars of white light 14 ft in length, placed at either 100- or 200-ft centers on the extended runway centerline. The 14-ft crossbars consist of closely spaced five-bulb white lights to give the effect of a continuous bar of light.

The high-intensity ALS is 2400 ft long (some are 3000 ft long) with various patterns of light located symmetrically about the extended

runway centerline and a series of sequenced high-intensity flashing lights located every 100 ft on the extended runway centerline for the outermost 1400 ft. In the high-intensity ALSs the 14-ft crossbars of five-bulb white light are placed at 100-ft intervals and in the mediumintensity ALSs these crossbars of white light are placed at 200-ft intervals both for a distance of 2400 ft from the runway threshold on the extended runway centerline. The high -intensity ALSs have a long crossbar of green lights at the edge of the runway threshold. The ALSF-2 system, shown in Fig. 8-5*a*, has two additional crossbars consisting

of three-bulb white light crossbars which are placed symmetrically about the runway centerline at a distance of 500 ft from the runway threshold and two additional three-bulb red light crossbars are placed symmetrically about the extended runway centerline at 100-ft intervals for the inner 1000 ft to delineate the edges of the runway surface. The ALSF -1 system, shown in Fig. 8- 5*b*, has two additional crossbars consisting of five-bulb red light crossbars which are placed symmetrically about the runway centerline at a distance of 100 ft from the runway threshold to delineate the edge of the runway and two additional three-bulb red light crossbars placed symmetrically about the extended runway centerline at 200 ft from the runway threshold.

The MALSR system, shown in Fig. 8-5*c*, is a 2400-ft mediumintensity ALS with runway alignment indicator lights (RAILs). The inner 1000 ft of the MALSR is the MALS portion of the system and the outer 1400 ft is the RAIL portion of the system. The system has sequential flashing lights for the outer 1000 ft of the system. It is recommended for category I precision approaches. The simplified short

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approach lighting system (SSALR) has the same configuration as the MALSR system.
At smaller airports where precision approaches are not required,
a medium ALS with sequential flashers (MALSF) or with sequenced flashers (MALS) is adequate. The system is only 1400 ft long compared
to a length of 2400 ft for a precision approach system. It is
therefore much more economical, an important factor at small airports.
the runway alignment indicator lights and these are only provided in the
outermost 400 ft of the 1400-ft system to improve pilot recognition of the runway approach in areas where there are distracting lights in the vicinity of the airport. The MALS system does not have the runway alignment indicator lights or the sequential flashers.
At international airports in the United States, the 2400-ft ALSs are often extended to a distance of 3000 ft to conform to international specifications.

Sequenced-flashing high-intensity lights are available for airport use and are installed as supplements to the standard approach lighting system at those airports where very low visibilities occur frequently. These lights operate from the stored energy in a capacitor which is discharged
through the lamp in approximately 5 ms and may develop as
much as 30 million cd of light. They are mounted in the same pedestals
as the light bars. The lights are sequence-fired, beginning with the unit

farthest from the runway. The complete cycle is repeated every 2 s. This
results in a brilliant ball of light continuously moving toward the runway.
Since the very bright light can interfere with the eye adaptation of

the pilot, condenser discharge lamps are usually omitted in the 1000 ft of the approach lighting system nearest the runway.

Visual Approach Slope Aids

Visual approach slope aids are lighting systems designed to provide a measure of vertical guidance to aircraft approaching a particular runway. The principle of these aids is to provide color-based identification to the pilot indicating their variation from a desired altitude and descent rate while on approach. The two most common visual approach slope aids are the visual approach slope indicator (VASI), and the precision approach path indicator (PAPI).

Visual Approach Slope Indicator

The visual approach slope indicator (VASI) is a system of lights which acts as an aid in defining the desired glide path in relatively good weather conditions. VASI lighting intensities are designed to be visible from 3 to 5 mi during the day and up to 20 mi at night.

There are a number of different VASI configurations depending on the desired visual range, the type of aircraft, and whether large wide bodied aircraft will be using the runway. Each group of lights transverse to the direction of the runway is referred to as a *bar*. The downwind bar is typically located between 125 and 800 ft from the runway threshold, each subsequent bar is located between 500 and 1000 ft from the previous bar. A bar is made up of one, two, or three light units, referred to as *boxes*. The basic VASI-2 system, illustrated in Fig. 8-6, is a two-bar system consisting of four boxes. The bar that is nearest to the runway threshold is referred to as the *downwind bar*, and the bar that is farthest from the runway threshold is referred to as the *upwind bar*. As illustrated in Fig. 8-6, if pilots are on the proper glide path, the downwind bar appears white and the upwind bar appears red; if pilots are too low, both bars appear red; and if they are too high both bars appear white.

In order to accommodate large wide bodied aircraft where the height of the eye of the pilot is much greater than in smaller jets, a third upwind bar is added. For wide bodied aircraft the middle bar becomes the downwind bar and the third bar is the upwind bar. In other words, pilots of large wide bodied aircraft ignore the bar closest to the runway threshold and use the other two bars for visual reference.. The more common systems in use in the United States are the VASI-2, VASI-4, VASI-12, and VASI-16. VASI systems are particularly useful on runways that do not have an instrument landing system or for aircraft not equipped to use an instrument landing system.

Precision Approach Path Indicator

The FAA presently prefers the use of another type of visual approach indicator called the *precision approach path indicator* (PAPI) [20]. This system gives more precise indications to the pilot of the approach path of the aircraft and utilizes only one bar as opposed to the minimum of two required by the VASI system.

The system consists of a unit with four lights on either side of the approach runway.

Threshold Lighting

During the final approach for landing, pilots must make a decision to complete the landing or ―execute a missed approach.‖ The identification of the threshold is a major factor in pilot decisions to land or not to land. For this reason, the region near the threshold is given special lighting consideration. The threshold is identified at large airports by a complete line of green lights extending across the entire width of the runway, and at small airports by four green lights on each side of the threshold. The lights on either side of

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the runway threshold may be elevated. Threshold lights in the direction
of landing are green but in the opposite direction these lights are red to indicate the end of the runway.

Runway Lighting

After crossing the threshold, pilots must complete a touchdown and roll out on the runway. The runway visual aids for this phase of landing
are be designed to give pilots information on alignment, lateral displacement, roll, and distance. The lights are arranged to form a visual pattern that pilots can easily interpret.

At first, night landings were made by floodlighting the general area. Various types of lighting devices were used, including automobile
headlights, arc lights, and search lights. Boundary lights were added to outline the field and to mark hazards such as ditches and fences. Gradually, preferred landing directions were developed, and special lights were used to indicate these directions. Floodlighting was then restricted to the preferred landing directions, and runway edge lights were added along the landing strips. As experience was

developed, the runway edge lights were adopted as visual aids on a runway. This was followed by the use of runway centerline and touchdown
zone lights for operations in very poor visibility. FAA Advisory Circular 150/5340-30C provides guidance for the design and installation
of runway and taxiway lighting systems.

Runway Edge Lights

Runway edge lighting systems outline the edge of runways during nighttime and reduced visibility conditions. Runway edge lights are classified by intensity, high intensity (HIRL), medium intensity (MIRL),

and low intensity (LIRL). LIRLs are typically installed on visual runways
and at rural airports. MIRLs are typically installed on visual runways at larger airports and on nonprecision instrument runways, HIRLs are installed on precision-instrument runways.
Elevated runway lights are mounted on frangible fittings and project no more than 30 in above the surface on which they are installed. They are located along the edge of the runway not more than

10 ft from the edge of the full-strength pavement surface. The longitudinal
spacing is not more than 200 ft. Runway edge lights are white, except that the last 2000 ft of an instrument runway in the direction of aircraft operations these lights are yellow to indicate a caution zone.

Runway Centerline and Touchdown Zone Lights

As an aircraft traverses over the approach lights, pilots are looking at relatively bright light sources on the extended runway centerline. Over the runway threshold, pilots continue to look along the centerline,
but the principal source of guidance, namely, the runway edge lights, has moved far to each side in their peripheral vision. The result is that the central area appears excessively black, and pilots are virtually

flying blind, except for the peripheral reference information, and any reflection of the runway pavement from the aircraft‘s landing lights. Attempts to eliminate this ―black hole‖ by increasing the intensity

of runway edge lights have proven ineffective. In order to reduce the black hole effect and provide adequate guidance during very poor visibility conditions, runway centerline and touchdown zone lights are typically installed in the pavement.
These lights are usually installed only at those airports which are equipped for instrument operations. These lights are required for ILS

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category II and category III runways and for category I runways used for landing operations below 2400 ft runway visual range. Runway centerline
lights are required on runways used for takeoff operations below 1600 ft runway visual range. Although not required, runway centerline

lights are recommended for category I runways greater than 170 ft in width or when used by aircraft with approach speeds over 140 kn. When there are displaced thresholds, the centerline lights are extended into the displaced threshold area. If the displaced area is not used for takeoff operations, or if the displaced area is used for takeoff operations and is less than 700 ft in length, the centerline lights are blanked out in the direction of landing. For displaced thresholds greater
than 700 ft in length or for displaced areas used for takeoffs, the centerline
lights in the displaced area must be capable of being shut off during landing operations.

Runway touchdown zone lights are white, consist of a three- bulb bar on either side of the runway centerline, and extend 3000 ft from the runway threshold or one -half the runway length if the runway is less than 6000 ft long. They are spaced at intervals of 100 ft, with the first light bar 100 ft from the runway threshold, and are located 36 ft on either side of the runway centerline, as shown in Fig. 8-13. The centerline lights are spaced at intervals of 50 ft. They are normally offset a maximum of 2 ft from the centerline to avoid the centerline paint line and the nose gear of the aircraft riding over the light fixtures.
These lights are also white, except for the last 3000 ft of runway

in the direction of aircraft operations, where they are color coded. The last 1000 ft of centerline lights are red, and the next 2000 ft are alternated

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red and white.

Runway End Identifier Lights

Runway end identifier lights (REIL) are installed at airports where there are no approach lights to provide pilots with positive visual identification of the approach end of the runway. The system consists of a pair of synchronized white flashing lights located on each side of the runway threshold and is intended for use when there is adequate visibility.

Taxiway Lighting

Either after a landing or on the way to takeoff, pilots must maneuver the aircraft on the ground on a system of taxiways to and from the terminal and hangar areas. Taxiway lighting systems are provided for taxiing at night and also during the day when visibility is very poor, particularly at commercial service airports.

The following overall guidance should be applied in determining the lighting, marking, and signing visual aid requirements for taxiways:

In order to avoid confusion with runways, taxiways must be clearly identified.

Runway exits need to be readily identified. This is particularly true for high-speed runway exits so that pilots can be able to locate these exits 1200 to 1500 ft before the turnoff point.

Adequate visual guidance along the taxiway must be provided.

Specific taxiways must be readily identified.

• The intersections between taxiways, the intersections between runways and taxiways, and runway-taxiway crossings need to be clearly marked.

The complete taxiway route from the runway to the apron

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and from the apron to the runway should be easily identified. There are two primary types of lights used for the designation of taxiways. One type delineates the edges of taxiways [21] and the other type delineates the centerline of the taxiway [27]. In addition, there is an increasing use of lighting systems on taxiways, such as runway guard lights (RGLs) and stop bars, to identify intersections with runways, in an effort to reduce accidental incursions on to active runway environments.

Taxiway Edge Lights

Taxiway edge lights are elevated blue colored bidirectional lights usually located at intervals of not more than 200 ft on either side of the taxiway. The exact spacing is influenced by the physical layout of the taxiways. Straight sections of taxiways generally require edge light spacing in 200-ft intervals, or at least three lights equally spaced for taxiway straight line sections less than 200 ft in length.

Closer spacing is required on curves. Light fixtures are located not more than 10 ft from the edge of full strength pavement surfaces.

Taxiway centerline lights are in-pavement bidirectional lights placed in equal intervals over taxiway centerline markings. Taxiway centerline lights are green, except in areas where the taxiway intersects with a runway, where the green and yellow lights are placed alternatively.

Research and experience have demonstrated that guidance from centerline lights is superior to that from edge lights, particularly in low visibility conditions.

For normal exits, the centerline lights are terminated at the edge of the runway. At taxiway intersections the lights continue across the intersection.

For long-radius high-speed exit taxiways, the taxiway lights are extended onto the runway from a point 200 ft back from the point of curvature (PC) of the taxiway to the point of tangency of the central curve of the taxiway. Within these limits the spacing of lights is 50 ft. These

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lights are offset 2 ft from the runway centerline lights and are gradually brought into alignment with the centerline of the taxiway.

Where the taxiways intersect with runways and aircraft are required to hold short of the runway, several yellow lights spaced at 5-ft intervals

are placed transversely across the taxiway.

Runway Guard Lights

Runway guard lights (RGLs) are in-pavement lights located on taxiways at intersections of runways to alert pilots and operators of airfield ground vehicles that they are about to enter onto an active runway. RGLs are located across the width of the taxiway, approximately 2 ft from the entrance to a runway, spaced at approximately 10-ft intervals,

Runway Stop Bar

Similar to runway guard lights, runway stop bar lights are in-pavement lights on taxiways at intersections with runways. As opposed to RGLs that provide warning to pilots approaching a runway, runway stop bar lights are designed to act as ―stop‖ lights, directing aircraft and vehicles on the taxiway not to enter the runway environment. Runway stop bar lights are activated with red illuminations during periods of runway occupancy or other instances where entrance from the taxiway to the runway is prohibited. In-pavement runway stop bar lighting is typically installed in conjunction with elevated runway guard lights located outside the width of the pavement.

Runway and Taxiway Marking

In order to aid pilots in guiding the aircraft on runways and taxiways, pavements are marked with lines and numbers. These markings are of benefit primarily during the day and dusk. At night, lights are used to guide pilots in landing and maneuvering at the airport. White is used

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for all markings on runways and yellow is used on taxiways and aprons.

Runways

The FAA has grouped runways for marking purposes into three classes: (1) visual, or –basic‖ runways, (2) nonprecision instrument runways, and (3) precision instrument runways. The visual runway is a runway with no straight-in instrument approach procedure and is intended solely for the operation of aircraft using visual approach procedures. The nonprecision instrument runway is one having an existing instrument approach procedure utilizing air navigation facilities with only horizontal guidance (typically VOR or GPS-based RNAV approaches without vertical guidance) for which a straight-in nonprecision approach procedure has been approved. A precision instrument runway is one having an existing instrument approach procedure utilizing a precision instrument landing system or approved GPS-based RNAV (area navigation) or RNP (required navigation performance) precision approach. Runways that have a published approach based solely on GPS-based technologies are known as GPS runways.

Runway markings include runway designators, centerlines, threshold markings, aiming points, touchdown zone markings, and side stripes. Depending on the length and class of runway and the type of aircraft operations intended for use on the runway, all or some of the above markings are required.

Runway Designators

The end of each runway is marked with a number, known as a runway designator, which indicates the approximate magnetic azimuth (clockwise from magnetic north) of the runway in the direction of operations. The marking is given to the nearest 10° with the last digit omitted. Thus a runway in the direction of an azimuth of 163° would be marked as runway 16 and this runway would be in the approximate direction of south-south-east. Therefore, the east end of an east-west runway

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would be marked 27 (for 270° azimuth) and the west end of an east-west runway would be marked 9 (for a 90° azimuth). If there are two parallel runways in the east -west direction, for example, these runways would be given the designation 9L -27R and 9R-27L to indicate the direction of each runway and their position (L for left and R for right) relative to each other in the direction of aircraft operations. If a third parallel runway existed in this situation it has traditionally been given the designation 9C -27C to indicate its direction and position relative (C for center) to the other runways in the direction of aircraft operations.

When there are four parallel runways, one pair is marked with the magnetic azimuth to the nearest 10° while the other pair is marked with the magnetic azimuth to the next nearest 10°. Therefore, if there were four parallel runways in the east-west direction, one pair would be designated as 9L-27R and 9R-27L and the other pair could be designated as either 10L-28R and 10R-28L or 8L-26R and 8R-26L. This type of designation policy is increasingly being applied to three parallel runway configurations, as well. For example, one pair would be designated as 9L-27R and 9R-27L and the third runway may be designated 10-28. Runway designation markings are white, have a height of 60 ft and a width, depending upon the number or letter used, varying from 5 ft for the numeral 1 to 23 ft for the numeral 7. When more than one number or letter is required to designate the runway the spacing between the designators is normally 15 ft. The sizes of the runway designator markings are proportionally reduced only when necessary due to space limitations on narrow runways and these designation markings should be no closer than 2 ft from the edge of the runway or the runway edge stripes.

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Runway Threshold Markings

Runway threshold markings identify to the pilot the beginning of the runway that is safe and available for landing. Runway threshold markings begin 20 ft from the runway threshold itself.

Runway threshold markings consist of two series of white stripes, each stripe 150 ft in length and 5.75 ft in width, separated about the centerline of the runway. On each side of the runway centerline, a number of threshold marking stripes are placed, For example, for a 100 -ft runway, eight stripes are required, in two groups of four are placed about the centerline. Stripes within each set are separated by 5.75 ft. Each set of stripes is separated by 11.5 ft about the runway centerline. The above specifications for runway threshold markings were adapted by the FAA from ICAO international standards and made mandatory for United States civil use airports in 2008.

Centerline Markings

Runway centerline markings are white, located on the centerline of the runway, and consist of a line of uniformly spaced stripes and gaps. The stripes are 120 ft long and the gaps are 80 ft long. Adjustments to the lengths of stripes and gaps, where necessary to accommodate runway length, are made near the runway midpoint. The minimum width of stripes is 12 in for visual runways, 18 in for nonprecision instrument runways, and 36 in for precision instrument runways. The purpose of the runway centerline markings is to indicate to the pilot the center of the runway and to provide alignment guidance on landing and takeoff.

Aiming Points

Aiming points are placed on runways of at least 4000 ft in length to provide enhanced visual guidance for landing aircraft. Aiming point markings consist of two bold stripes, 150 ft long, 30 ft wide, spaced

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72 ft apart symmetrically about the runway centerline, and beginning 1020 ft from the threshold.

Touchdown Zone Markings

Runway touchdown zone markings are white and consist of groups of one, two, and three rectangular bars symmetrically arranged in pairs about the runway centerline. These markings begin 500 ft from the runway threshold. The bars are 75 ft long, 6 ft wide, with 5 ft spaces between the bars, and are longitudinally spaced at distances of 500 ft along the runway. The inner stripes are placed 36 ft on either side of the runway centerline. For runways less than 150 ft in width, the width and spacing of stripes may be proportionally reduced. Where touchdown zone markings are installed on both runway ends on shorter runways, those pairs of markings which would extend to within 900 ft of the runway midpoint are eliminated.

Side Stripes

Runway side stripes consist of continuous white lines along each side of the runway to provide contrast with the surrounding terrain or to delineate the edges of the full strength pavement. The maximum distance

between the outer edges of these markings is 200 ft and these markings have a minimum width of 3 ft for precision instrument runways

and are at least as wide as the width of the centerline stripes on other runways.

Displaced Threshold Markings

At some airports it is desirable or necessary to ―displace‖ the runway threshold on a permanent basis. A displaced threshold is one which has been moved a certain distance from the end of the runway. Most often this is necessary to clear obstructions in the flight path on landing.

The displacement reduces the length of the runway available for landings, but takeoffs can use the entire length of the runway. T.

These markings consist of arrows and arrow heads to identify the displaced

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threshold and a threshold bar to identify the beginning of the runway threshold itself. Displaced threshold arrows are 120 ft in length, separated longitudinally by 80 ft for the length of the displaced

threshold. Arrow heads are 45 ft in length, placed 5 ft from the threshold bar. The threshold bar is 5 ft in width and extends the width of the runway at the threshold.

Blast Pad Markings

In order to prevent erosion of the soil, many airports provide a paved *blast pad* 150 to 200 ft in length adjacent to the runway end. Similarly, some airport runways have a *stopway* which is only designed to support aircraft during rare aborted takeoffs or landing overruns and is not designed as a full strength pavement. Since these paved areas are not designed to support aircraft and yet may have the appearance of being so designed, markings are required to indicate this.

Centerline and Edge Markings

The centerline of the taxiway is marked with a single continuous 6-in yellow line. On taxiway curves, the taxiway centerline marking continues from the straight portion of the taxiway at a constant distance from the outside edge of the curve. At taxiway intersections which are designed for aircraft to travel straight through the intersection, the centerline markings continue straight through the intersection. At the intersection of a taxiway with a runway end, the centerline stripe of the taxiway terminates at the edge of the runway.

At the intersection between a taxiway and a runway, where the taxiway serves as an exit from the runway, the taxiway marking is usually

extended on to the runway in the vicinity of the runway centerline marking. The taxiway centerline marking is extended parallel to the

runway centerline marking a distance of 200 ft beyond the point of tangency. The taxiway curve radius should be large enough to provide a clearance to the taxiway edge and the runway edge of at least one - half the width of the taxiway. For a taxiway crossing a runway, the taxiway centerline marking may continue across the runway but it must be interrupted for the runway markings.

When the edge of the full strength pavement of the taxiway is not readily apparent, or when a taxiway must be outlined when it is

established on a large paved area such as an apron, the edge of the taxiway is marked with two continuous 6-in wide yellow stripes that are 6 in apart.

Taxiway Hold Markings

For taxiway intersections where there is an operational need to hold aircraft, a dashed yellow holding line is placed perpendicular to and across the centerline of both taxiways.

When a taxiway intersects a runway or a taxiway enters an instrument landing system critical area, a holding line is placed across the taxiway. The holding line for a taxiway intersecting a runway consists of two solid lines of yellow stripes and two broken lines of yellow stripes placed perpendicular to the centerline of the taxiway and across the width of the taxiway. The solid lines are always placed on the side where the aircraft is to hold. The holding line for an instrument
landing system critical area consists of two solid lines placed

perpendicular to the taxiway centerline and across the width of the taxiway joined with three sets of two solid lines symmetrical about and parallel to the taxiway centerline.

Taxiway Shoulders

In some areas on the airfield, the edges of taxiways may not be welldefined
due to their adjacency to other paved areas such as aprons
and holding bays. In these areas, it is prudent to mark the edges of

taxiways with shoulder markings. Taxiway shoulder markings are yellow in color, and are often painted on top of a green background. The shoulder markings consist of 3-ft-long yellow stripes placed perpendicular
to the taxiway edge stripes, On
straight sections of the taxiway, the marks are placed at a maximum spacing of 100 ft. On curves, the marks are placed on a maximum of 50 ft apart between the curve tangents.

Distances shown above are for planning purposes only. ―Hold position markings‖
must be placed in order to restrict the largest aircraft (tail or body) expected
to use the runway from penetrating the obstacle-free zone.
For aircraft approach categories A and B, airplane design group III, this distance
is increased 1 ft for each 100 ft above 5100 ft above sea level. For airplane design
group IV, precision instrument runways, this distance is increased 1 ft for each
100 ft above sea level.
For aircraft approach category C, airport design group IV, precision instrument
runways. This distance is increased 1 ft for each 100 ft above sea level. For airplane
design group V, this distance is increased 1 ft for each 100 ft above sea level.
For aircraft approach category D, this distance is increased 1 ft for each 100 ft
above sea level.

Enhanced Taxiway Markings

Beginning in 2008, all airports serving commercial air carriers are required to mark certain critical areas of the airfield with enhanced taxiway markings. These markings are designed to provide additional

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guidance and warning to pilots of runway intersections. Enhanced markings consist primarily of yellow-painted lines, using paint mixtures with imbedded glass beads to enhance visibility. In addition, yellow markings must be marked on top of a darkened black background.

Taxiway centerlines are enhanced for 150 ft from the runway hold-short markings. The centerline enhancements include dashed yellow lines 9 ft in length, separated longitudinally by 3 ft. These yellow lines are placed 6 in from each end of the existing centerline.

Closed Runway and Taxiway Markings

When runways or taxiways are permanently or temporarily closed to aircraft,
yellow crosses are placed on these trafficways. For permanently

closed runways, the threshold, runway designation, and touchdown markings are obliterated and crosses are placed at each end and at 1000 ft intervals. For temporarily closed runways, the runway markings
are not obliterated, the crosses are usually of a temporary type and are only placed at the runway ends. For permanently closed taxiways,
a cross is placed on the closed taxiway at each entrance to the taxiway. For temporarily closed taxiways barricades with orange and white markings are normally erected at the entrances.

Airfield Signage

In addition to markings, signage is placed on the airfield to guide and direct pilots and ground vehicle operators to points on the airport. In addition some signage exists to provide the pilots with information regarding their position on the airfield, the distance remaining on a runway, the location of key facilities at the airport, and often informative
signage ranging from voluntary procedures to mitigate noise impacts to warnings about nearby security sensitive areas

Runway Distance Remaining Signs

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Runway distance remaining signs are placed on the side of a runway and provide the pilot with information on how much runway is left during takeoff or landing operations. These signs are placed at 1000 ft intervals along the runway is a descending sequential order.
In this configuration it is recommended that the signs be placed on the left side of the most frequently used direction of the runway.
The signs may be placed on the right side of the runway when necessary
due to required runway-taxiway separations or due to conflicts between intersecting runways or taxiways. An alternative method is to provide a set of single-faced signs on either side of the runway to indicate the distance remaining when the runway is used in both directions. The advantage of this configuration is that the distance remaining is more accurately reflected when the runway length is not an even multiple of 1000 ft. Another alternative uses double-faced signs on both sides of the runway. The advantage of this method is that the runway distance is displayed on both sides of the runway in each direction which is an advantage when a sign on one side needs to be omitted because of a clearance conflict. When the runway distance

is not an even multiple of 1000 ft, one half of the excess distance is added to the distance on each sign on each runway end.

Taxiway Guidance Sign System

The primary purpose of a taxiway guidance sign system is to aid pilots
in taxiing on an airport. At controlled airports, the signs supplement the instructions of the air traffic controllers and aid the pilot in complying
with those instructions. The sign system also aids the air traffic controller by simplifying instructions for taxiing clearances, and the routing and holding of aircraft. At locations not served by air traffic control towers, or for aircraft without radio contact, the sign system provides guidance to the pilot to major destinations areas in the airport.

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The efficient and safe movement of aircraft on the surface of an airport requires that a well-designed, properly thought-out, and standardized taxiway guidance sign system is provided at the airport. The system must provide the pilot with the ability to readily determine the designation of any taxiway on which the aircraft is located, readily identify routings to a desired destination on the airport property, indicate mandatory aircraft holding positions, and identify the boundaries for aircraft approach areas, instrument landing system critical areas, runway safety areas and obstacle free zones. It is virtually impossible, except for holding position signs, to completely specify the locations and types of signs that are required on a taxiway system at a particular airport due to the wide variation in the types of functional layouts for airports.

Taxiway Designations

Taxiway guidance sign systems are in a large part based on a system of taxiway designators which identify the individual taxiway components.

While runway designators are based on the magnetic heading of the runway, taxiway designators are assigned based on an alphabetic ordering system, independent of the taxiways direction of movement. Taxiways are typically identified in alphabetic order from east to west or north to south (i.e., the northern or easternmost taxiway would be designated ―A‖, the next southern or western taxiway would be designated ‗B,‖ and so forth). Entrance and exit taxiways perpendicular to main parallel taxiways are designated by the letter of the main parallel taxiway from which they spur, followed by a numeric sequence. For instance, the northernmost entrance taxiway off of taxiway ‗A‖ would be designated ‗A1,‖ and so forth. The letters ―I‖ and ―O‖ are not used as taxiway designators due to their similarity in form

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to the numbers ―1‖ and ―0.‖ In addition the letter ―X‖ is not used as a taxiway designator due to its similarity to a closed runway marking. An example taxiway designation scheme is illustrated in Fig. 8-30.

The taxiway guidance sign system consists of four basic types of signs: *mandatory instruction* signs, which indicate that aircraft should not proceed beyond a point without positive clearance, *location* signs, which indicate the location of an aircraft on the taxiway or runway system and the boundaries of critical airfield surfaces, *direction* signs, which identify the paths available to aircraft at intersections, and *destination*

signs, which indicate the direction to a particular destination.

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