25

Railways

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25.1 Earthworks and drainage

The contours of the territory to be crossed by a railway are obviouslv decisive as to its average gradient but they are also the background to fixing the maximum permissible gradient within the limits of tractive and braking adhesion. The lower the difference between average gradient and maximum gradient the greater is the practicable train load and the lesser are the deviations from constant-speed running. The minimum curvature to be used also determines the differences between the line speed limit and local speed restrictions. Long curves of small radius on heavy gradients may involve derailment hazards for very long and heavy trains, arising from braking or tractive effort surges along the train. Both maximum gradient and minimum curvature have a large effect on the earthworks cost of constructing a railway and, because of this, the ideal of constant-speed running is often subject to heavy qualification. This is particularly the case in mountainous country.

Railway alignment needs to be planned to give a volume balance between excavation in cuttings and tipping in embankments, subject to the material excavated being suitable for tipping to form embankments and subject to minimizing the haul of the excavated soil. Recourse to borrow pits for embankments and spoil hauls for cuttings should be minimized.

The route may also be affected by other considerations which may be economic, environmental or technical. These would become apparent in a full site investigation, where such matters as previous mining activity, underground services, effect on neighbouring structures, nature of the groundwater table and watercourses would be taken into account. Guidance is available in relevant Codes of Practice, such as BS 6031 and BS 5930. (See also Chapters 9, 11 and 17.)

25.1.1 Site investigation

This will vary according to the extent of the problem. At the outset, a preliminary study may give adequate information to specify the route corridor from geological maps and memoirs, topographical maps and aerial photographs. Earth satellite imagery with interpretation of selective wavebands by specialists in remote sensing can indicate important features. Water table conditions may vary throughout the year from those obtaining at the time of exploration.

For more localized investigation, the type of equipment (augers, percussion and rotary tools, penetration heads, loading plates, pumps), instruments (piezometers, inclinometer tubes, seismometers, resistivity meters, gravimeters, etc.) must be chosen according to conditions. Relevant disturbed or undisturbed samples should be procured for testing. According to the type of ground, the construction and the design philosophy applied, it may be necessary to carry out full-scale site testing with long-term monitoring of instruments.

25.1.2 General

In the past, railways have been maintained over poor ground, with inadequate trackbed materials with a high input of labour time and at slow or moderate train speeds. Although the following concepts may be used in modifying old railways, the basic approach is to obtain a minimum maintenance high-speed railway accepting normal freight traffic on conventional sleepers.

25.1.2.1 Cutting slopes

Slopes in natural ground may be constructed at safe angles according to the properties of the soil.

25.1.2.2 Rock cuttings

British Railways has indicated safe slopes for rock cuttings and angles of repose for rock embankments in a chart reproduced as Table 3 in BS 6031. If steep cuttings in rock are essential, then it is necessary to apply engineering geology concepts to assess joint sets in relation to the direction of slope. Rock anchors, rock bolts and sprayed concrete may be used to assure stability before considering the use of mass-retaining walls. Chalk and certain other soft rocks are susceptible to weathering and frost action and may be protected by vegetation cover.

25.1.2.3 Soil cuttings

Where the slopes are in non-cohesive sands and gravels, the angle of repose is the limiting gradient (with a maximum value of 1:1). Usually, the gradient is shallower than this due to the presence of finer layers in the soil system or a silt or clay matrix around the gravel. The non-cohesive soils tend to be self-draining but erosion can occur when water springs part way up the slope. At such locations a non-woven geotextile filter can be placed and covered with uniform coarse stone (away from the sun's rays) which will hold the fabric in place.

Drainage measures should take the form of preventing water reaching the slope and of removing it from the slope. Unlined ditches behind the crest of the slope increase the hazard. Drainage trenches, whether behind or below the crest, should be designed to intercept water and have impermeable membranes below them and on the downfill face to prevent water once collected from re-entering the soil. Modern counterfort or buttress drains differ greatly from the original open forms. Like all drains they should be lined with a geotextile layer to prevent erosion behind and fouling within them. The top 1 or 2 m should be composed of impermeable material, either compacted clay fill or a system of stone and plastics membrane to prevent surface water reaching deep into the ground, where it could cause internal pore water pressure at likely slip surfaces.

In some cases, there is a mantle of more permeable silt or sand overlying the older clay and, if possible, a lateral interceptor drain should be placed about 20 m behind the crest to prevent the fast seepage of water.

In addition to the above rotational slides, translational slides can occur, usually as a shallow mass moving on a planar surface. They can take the form of slab or block slides, wedge failures, debris slides and flow slides. These possibilities are assessed in the site investigation. Mires are formed as peat is laid down in a specific sequence with variations in soil content, dimensions of fibre (roots, trees, etc.) and extent of humification. Peats reduce greatly in volume under the effects of loading and drainage. In making cuttings in a peat system, quite substantial waterflow can occur and a filter is advised to deal with fine particles otherwise carried from the slope.

In all cases of cuttings, other engineering works at ground level should be considered. Urban or industrial construction involves roads which act as catchments to deliver water to local drainage systems and also water services: if these are defective and near the slope, water can flow to increase pore pressure and precipitate a slip. Similarly, surcharging, especially if accompanied by dynamic loads from construction plant or the placing of storage containers, can seriously reduce the factor of safety.

Steep rock faces, chalk cliffs, and boulder-strewn hill slopes may present rockfall or chalkfall problems which may need special watchmen, signalling provisions, special fence, apron or tunnel protection.

25.1.2.4 Embankments

The nature of the natural soil to receive the embankment must be investigated. If it is too weak to receive the embankment loading at the rate of placing likely to be used by the contractor, failure could occur. A total stress analysis is applicable for this. The conventional technique is to place berms as counterweights at the position of heave. A modern alternative is the use of reinforcing geotextiles or meshes (usually of plastics) to resist tensile forces. The fabric is laid directly on the ground and covered with a granular layer at least 200 mm thick; this acts as a filter and permits construction plant to move easily over the site without sinking into the underlying soft soil. Further fabric or mesh is laid at higher levels, the number and spacing of layers depending upon the engineering properties of the specialist type of material chosen.

Slope angles. Embankments may be formed at angles varying from ratios of 1:1 horizontal to vertical for crushed rock and gravels to 2.5:1 or even shallower for clays and silts. The slope angles depend both on the material and rainfall. For modern railways, peat should not be used as fill material.

The surfaces of slopes should be protected from erosion. This may occur naturally as vegetation is established, followed by a protective topsoil. If surface erosion is a problem, various systems are available, such as spraying with a seed mulch, turving, placing filter fabrics held down by gravel layers or using a honeycomb mesh to hold seeded compost in place.

Rockfalls. Vertical or sloping rock faces may erode or topple to cause rocks of various sizes to fall towards the running line. Although vegetation may help by bonding superficially, protection must involve coping with the energy of rockfall and removing the debris regularly. If space is available, one or two berms are constructed between the base of the slope and the railway to collect scree. For some faces, a plastic geomesh is adequate.

Reinforced earth. The dimensions of embankments and of gravity-retaining structures can be reduced by the inclusion of various strengthening meshes, fabrics, strips and rods of metal, glass fibre or plastics. The tensile resistance of these elements is applied to the adjacent granular soil to produce a composite system permitting the construction of vertical faces in the fill. These external faces are protected by facing elements usually of concrete, resulting in a structure which is economic and which can accommodate settlement. The various qualities of creep, longevity of reinforcement, corrosion, etc. are still the subject of study but many such structures exist throughout the world, including railway environments. There are none so far beneath a high-speed running line and, in this particular case, such systems can be installed up to a horizontal distance of 5 m from the running line.

Trackbed designs. The thickness of subsleeper construction (trackbed) is a function of number and size of axle loadings and of the subgrade soil. Modern railways assess the subgrade soil in one of two approaches: (1) the classification of the soil according to its physical properties and taking account of the water table; or (2) correlating some measured strength or modulus of the subgrade with an empirical design chart. The thickness of the trackbed may also depend upon its component layers to protect against frost, water and particle movement. In very frost-prone areas, such as Central Europe, the thickness of ballast for frost protection exceeds that which might be required for prevention of bearing capacity failure.

The thickness of ballast (all dimensions are below bottom of sleeper) is a minimum of 200 mm to permit tamping machines to operate. Although new ballast injection machines would permit this to be reduced, a high-speed or high-axle-load railway would require 300 to 500 mm thickness for minimum maintenance.

Drainage. The control of water in the trackbed is a major factor in designing the construction layers in relation to the type of subgrade soil as discussed above. Soils such as sands and gravels which may be drained fairly easily do not present a great problem unless there is an artesian head: in this case, there can be a slow upward migration of fine or medium sand under track vibration combined with water flow, and a geotextile is necessary to hold down this sand. Cohesive soils cannot be drained easily and the installation of a channel or pipe will only reduce the pore water pressure to invert level in its immediate vicinity. It is not practical to attempt to remove water from clay in this way as such a large number of drains would be required. If water arrives through precipitation or by flow from adjacent areas, then a relatively small amount is sufficient to produce deleterious changes in pore pressure and so it is practical to provide drains to intercept and remove this free water. Most of the water in cohesive soil is held in capillary suction.

The relationship between the moisture content of cohesive soil and the water table is complex, depending upon the over consolidation ratio (OCR) of the soil. Weathering reduces the OCR effect at the surface.

One object in designing the system of drains and track is to produce a maintenance-free system or one needing minimal attention. Channel drains are readily accessible for cleaning and deal with rainwater, they can be laid at very slight gradients and can deliver at catchpits to deeper pipe carriers if necessary. Many pipe drains act both as collectors and carriers, allowing water to enter at open joints or through perforations. The various forms of pipe are glazed earthenware, galvanized corrugated steel, pitch fibre and, now coming more into use because of ease of handling, plain or perforated PVC pipe. Geotextiles are of great use in static drainage conditions and most railways report satisfactory results using commercially available filtering non-woven fabric. For normal purposes, a fully heat-bonded geotextile with a surface density in the range 100 to 200 g/m^2 is acceptable; needle-punched geotextiles should be of slightly heavier grade. Fabrics placed in quasi-static conditions near the track should be of heavier grade up to 350 g/m^2, or more if needle-punched. Such a geotextile would be placed in the sixfoot of a double track line after one track had been cleaned or blanketed. This would prevent slurry from the dirty adjacent track flowing across.

A modern standard design for a side drain is a trench lined with geotextile with a perforated pipe drain running along its base; above and around the pipe is placed uniform stone such as ballast, with the top of the geotextile lapped over the stone about 200 mm below the surface: more stone on the geotextile protects it from disturbance and from the effect of the ultraviolet rays of the sun. As perforated drains can release as well as collect water, possibly at susceptible locations, it is often the practice to place a polyethylene film to line the trench.

25.2 Ballast

Ballast, the material around and below the sleeper, is placed to provide support and lateral resistance to the sleeper. It permits adjustment of level and alignment as required and, if this is done manually, the maximum size of particles should be about 50 mm. Ballast should be free-draining, mainly of single size, of cubical shape but, above all, durable so that there is negligible volume change under track loading. The wet attrition value (WAV) gives the best correlation, with minimum maintenance requirements over the long term and this is determined by the test described in BS 812:1951, clause 27, which specifies the exact size and type of sample to be used. If particles of different size from the 50 to 37.5 mm required in the test are used, then a different WAV is obtained. The WAV should not exceed 4% for good ballast and it is possible to obtain stone with a WAV down to 1%. It is rare for synthetic stone, such as slag, to have adequate wet attrition properties and they may generally be grouped with most limestones as being unsuitable to be in contact with the sleeper. Hardness is difficult to define in relation to other tests but, if the ultrasonic pulse velocity of the homogeneous mineral is 6000 m/s or greater, the stone will be suitable. In severe climates, a freezing and thawing test may be applicable.

The dimensions should conform by weight to the values shown in Table 25.1; and the 1.18 mm limit effectively minimizes the amount of dust present.

Table 25.1

Square mesh sieve	*% to pass*
63	100
50	100–97
28	20–0
14	2–0
1.18	0.3–0

The stone should have a maximum flakiness index of 50%. For elongation qualities, not more than 2% by weight of particles should have a dimension exceeding 75 mm.

The effect of many tamping cycles is to break up ballast particles; stone as hard as possible is required to accommodate this. When a sleeper is tamped, a horizontal load – the major principal stress – is applied to the ballast, causing it to deform vertically and lift the sleeper. The minor principal stress is vertical with the subsequent arrangement of stone particles in the least favourable position to support track loading, even though the rail level is now correct. The maximum rate of rail settlement occurs after tamping, which reduces as the stone packs down under traffic, with the major principal stress becoming vertical. The trackbed becomes more stable under vertical loading and the best relevelling procedure is the manual or mechanical placing of measured quantities of small stones of nominal 20 mm size between the ballast bed and the sleeper, which is lifted to insert the stone. There is now a track-levelling machine, electronically controlled, which evaluates cant and level from an advancing inclinometer trolley and places the exact quantity of stone by pneumatic injection to obtain proper level. As stone more than 50 mm below the sleeper is not moved and brought with its dirty matrix up to bottom sleeper level, as with a tamper, the likelihood of pumping track is somewhat lessened; the beneficial effect of high-quality small stone is not reduced by mixing with existing worn stone and there is less attrition of the base of the sleeper.

25.2.1 Track profile

Under traffic, rail level, as measured by accurate optical or inclinometer-based machines, shows a profile which is repeated even after many successive tamping and loading cycles. This is related to the care with which the original trackbed layers were installed. When the subgrade is prepared initially, and as subsequent layers of blanket and ballast are placed and compacted, it is necessary to provide extra sighting instruments on compaction and grading plant so that there are no short variations in level. This can be successfully done by using a laser system aimed at the surface a fixed distance ahead and which causes the equipment to compensate for deviation from the required level by moving its instrumented blade up or down.

The ends of the sleepers need to be boxed-in with shoulder ballast to a minimum width of 150 mm for any tracks, 200 to 250 mm for running lines carrying moderate-speed, moderate-axle-load traffic, 300 mm minimum for welded track on the straight and 350 mm for welded track on curves. Generally, there is little advantage in extending shoulder ballasting beyond 300 to 350 mm. In the last decade or so, the practice of raising the shoulder ballast in a slope from about top of sleeper level at the rail to about top of rail level at the shoulder edge has become common on European railways. This practice not only increases the lateral stability of the track but provides a useful reserve of boxing ballast which can be temporarily utilized to make good the boxing ballast when the track is tamped. The angle of the shoulder should be about 55°.

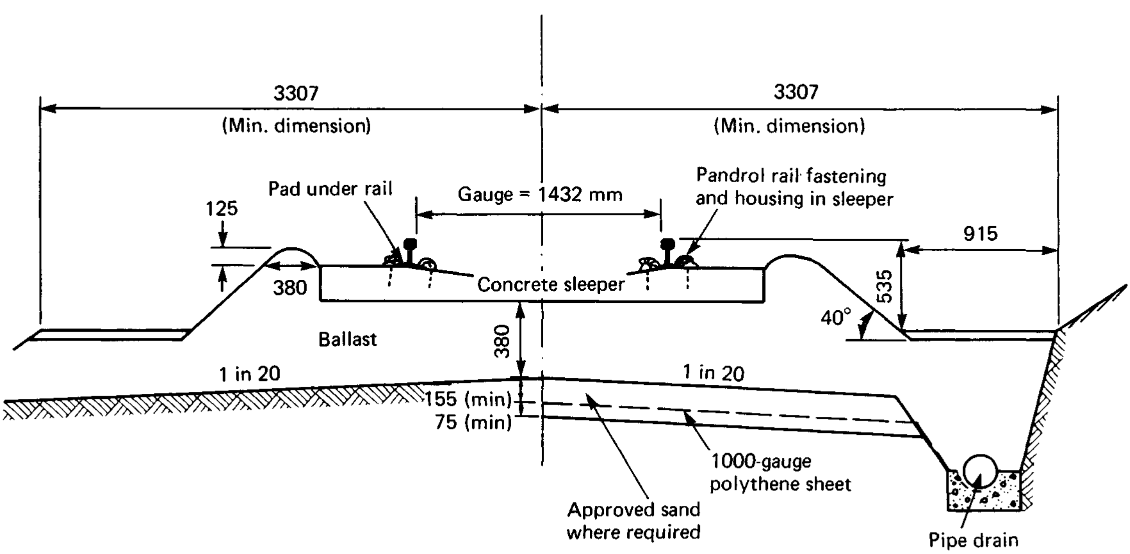


Figure 25.1 Components and formation for continuous welded rail (CWR) track (dimensions in millimetres)

25.3 Sleepers

25.3.1 Timber sleepers and sleeper spacing

The traditional track support since the early days of railways has been the timber sleeper, but now prestressed concrete sleepers are in use in many countries. In some countries the baseplated timber sleeper is already more expensive to install than a baseplateless prestressed concrete sleeper.

In Britain, the sleepers and point and crossing timbers have traditionally been of softwood, chiefly douglas fir from Canada, Maritime pine from southwest France and Corsica, and Baltic redwood from Poland and Russia. Homegrown fir sleepers have been used when they have been available. In continental Europe, beech and oak sleepers have been widely used. All softwood sleepers, and most hardwood sleepers that can be impregnated, are pressure creosoted by the Bethell or Ruping processes before being baseplated. The harder softwood sleepers such as douglas fir or scots fir and hardwood sleepers need to be incised prior to creosoting. Four to 14 litres of creosote per sleeper is a normal absorption, according to species. In Canada, the US and South America a mixture of mineral oil and creosote is used.

A timber sleeper may have a first or running line life of between 6 and 50 years with an average of about 20 years according to traffic loading, weather exposure, the nature and incidence of the indigenous vegetative and insect enemies of timber, the presence or absence of baseplates, the nature of the fastenings, the quality of the ballast and its maintenance, the species of timber and, significantly, the rate at which it has grown.

The British softwood timber sleeper is 250 × 125 × 2600 mm with switch and crossing timbers of 300 × 150 mm section. In Europe, hardwood sleepers mainly of oak or beech are used with a depth of 160 mm. Sleeper spacing in Britain is 700 mm with 650 mm on curves or on timber sleepered continuous welded rail (CWR). In countries other than Britain, timber sleeper spacings of 600 mm are common with some as close as 550 mm in heavy-axle situations.

The sleeper spacing applied represents a compromise between overall sleeper cost and the lessened frequency of maintenance attention and reduced rail bending stress which results therefrom. In addition, a decrease in sleeper spacing may be appropriate where the formation is weak or where it is not practicable to increase the total track construction depth by increasing the depth of ballast. Since also increasing the density of sleepering increases both the lateral and vertical resistances to buckling movements of the track, it is general practice to reduce sleeper spacing when laying track intended to carry long welded rails. Where the rail formation is regularly subjected to frost heave distortions, there may be a case for deliberately choosing a rather weak rail with complementary close sleeper spacing in order to allow the track to accommodate itself to a frost heave contour without imposing undue bending stresses in the rails.

25.3.2 Steel sleepers

Steel sleepers are widely used in India, Africa, South America, Asia and in parts of Europe. Steel shortages during and following the Second World War, and the increased use of concrete, impeded the further development of steel sleepers until about the mid 1970s. Since then, there has been a considerable amount of research done in Australia, Europe, Japan and Britain to evolve improved steel sleeper designs. Computer-based design techniques, e.g. finite element analysis, and modern stress measuring and analysis techniques have been applied to both full-scale laboratory loading and fatigue testing and to in-track service trials in order to prove the efficacy of new steel sleeper designs.

Many of the older steel sleeper designs gave average service lives of over 50 years before failing through cracking in, or adjacent to, the rail seat area. Generally, these cracks propagated from rail fastener holes or slots or from discontinuities in the rail seat area. Abrasive wear and/or corrosion in this area also contributed ultimately to sleeper failure. The steel sleeper design features, and especially those associated with the rail fastener mountings, together with the accumulated gross traffic tonnage carried, are the most important factors determining steel sleeper life. Recently improved steel sleeper designs have given due cognizance to these influences.

It is commonplace to recondition old steel sleepers which have already given a 30- to 50-year 'first life', by welding-on suitably designed reconditioning plates. If required, the reconditioning plates used may permit a change in rail fastener type or rail section to be made for the further 'second life' of the steel sleeper. When damaged in service, e.g. by derailments, steel sleepers can readily be repaired by pressing and/or welding.

Steel sleepers do not burn or suffer from exposure to dry heat. In tropical climates their immunity to insect or fungoidal damage is extremely beneficial.

Loss of metal section through corrosion is usually surprisingly low, but there may be a few very special sites subject to severe corrosion, where the use of steel sleepers would be inadvisable. Under-design of the rail seat, or inadequate support thereof, can greatly increase the localized corrosion and fatigue damage in this generally highly stressed area, and due allowances have been made in modern designs.

Steel sleepers pack neatly into bundles and thus simplify all handling and transport operations and greatly reduce these costs. The relatively low mass and convenient and uniform shape of steel sleepers is of benefit in both mechanical and manual handling.

The use of steel sleepers provides good resistance to lateral and longitudinal movement of the track and gives high consistency of gauge, both at installation and during subsequent service. The achievement of good alignment is necessary when steel sleepers are initially installed and when retamping after a brief 'running in' period. The additional care and attention at this early stage generally results in a reduced need for subsequent maintenance, realignment and retamping. Some of the older steel sleeper designs were difficult to install and adjust, but modern designs are now relatively easy to install and to use for 'spot replacements'.

The inverted trough shape and its entrapment of tamped-up ballast provides many benefits. The natural resistance of the ballast is more effectively utilized by steel sleepers, and ballast depth may be decreased by 75 to 100 mm as compared with wood or concrete sleepers. This can be of distinct advantage in tunnels with limited clearances and ballast depth. Where standard ballast depth is retained, the additional load-spreading effect of steel sleepers and their ballast-filled inverted trough, may enable sleeper spacing to be slightly increased. Also, the inverted trough shape has the in-built benefit that any enforced movement of the sleeper increases the entrapped ballast density and its resistance to further sleeper movement, in comparison with the solid shape of wood and concrete sleepers where eventually a 'break-free' point is reached.

Steel sleepers are similar to concrete in their tendency to degrade soft ballast more rapidly than wooden sleepers. Hence, the use of a hard stone is preferred for steel sleepers. Nevertheless, many old steel sleepers have given long and satisfactory service utilizing a gravel-type ballast.

Insulating pads, fastener shoes and/or washers must be incorporated in the steel sleepers designed for lines using track circuit signalling. In most cases the insulating components used are identical to those used with concrete sleepers. Modern plastic materials have improved the electrical security of these insulating components and the track circuits, but the high electrical conductivity of steel sleepers requires special attention to their design and maintenance.

The mass of individual steel sleepers is usually mid-way between that of wood and concrete, and may be considered a disadvantage with respect to resistance to the vertical buckling of continuously welded rail track. However, the entrapped ballast in the inverted trough of steel sleepers and their general shape (particularly the newer designs) greatly increases the 'effective in-track mass' of steel-sleepered track.

Most rail fastener types can be accommodated in steel sleeper designs, and there are only isolated fastener systems which cannot be economically incorporated in steel sleeper designs.

25.3.3 Concrete sleepers

The first experiments with reinforced concrete sleepers were made over 100 years ago. Concrete is attractively resistant to decay, insect attack and fire, but it is brittle, and reinforced concrete sleepers would not withstand the impact loads in main line track. A solution developed in France in the 1920s was to use pairs of reinforced concrete blocks connected by steel tie bars. This twin-block sleeper continues in use notably in France, Spain and North Africa but is of declining importance worldwide.

Under the pressures of timber shortages during the Second World War, a satisfactory monoblock concrete sleeper was developed in Britain in 1943 using the then new technique of pretensioned, prestressed concrete. This has led to a major industry in Britain (over 30 million pretensioned concrete sleepers have been supplied to British Rail) and many other countries (notably Norway, Sweden, USSR, Hungary, Czechoslovakia, Iraq, Japan, South Africa, Australia, Canada and the US).

In Britain and most of the other countries pretensioned concrete sleepers are made by the long-line method in which the tendons are fully bonded. This gives good distribution and control of prestress. Post-tensioned sleepers were developed in West Germany in the 1950s and have found limited favour (in West Germany, Italy, Spain and Mexico).

The current British Rail standard sleeper designated F27BS, is prestressed with six 9.3 mm, seven-wire strands (Figure 25.2). It is 2.515 m long, 203 mm deep under the rail, 264 mm wide at the base and weighs 280 kg. For more arduous conditions, a stronger version F27AS has been introduced, prestressed with eight strands in the same concrete envelope as the F27BS. Recently the F40, a 2.480 m long sleeper, has been developed to facilitate single-line mechanized track renewal within the tight British Rail structure gauge. The base width is increased to 285 mm to maintain the bearing area and the depth is 200 mm under the rail which gives the same weight as the F27BS. Prestressing is by six strands.

Spacing in British track is usually 700 or 650 mm, although 600 mm spacing is used in severe curves.

In the past 15 years, concrete sleepers have gained wider international acceptance for several reasons, including:

(1) Higher passenger train speeds and heavier freight axle loads necessitate higher quality and stronger track.
(2) High labour costs and intensive track use necessitate reduced frequency of maintenance cycles.
(3) Increased use of mechanized track-laying and renewal equipment has overcome the difficulty of handling heavy concrete sleepers.
(4) Softwood sleepers are often unable to withstand the stresses in modern main-line track and good-quality hardwood is increasingly expensive and difficult to obtain.
(5) With modern fastenings, only a resilient pad is required between the rail and sleeper thereby eliminating expensive base plates.

It is generally predicted that prestressed concrete sleepers will have an average life of 40 to 50 years. Some of the earlier designs of concrete sleepers have been removed from the track due to premature failure of the fastenings although the concrete remains structurally sound. Since 1964, British Rail have used the Pandrol fastening in which the less durable components can be renewed (Figure 25.3). At high speeds (over 110 km/h) wheel and rail surface defects can cause high stresses in concrete sleepers which, if allowed to persist, can cause cracking in the rail seat area of sleepers. Prestressed sleepers with such cracks can remain serviceable, providing the cause of high stress is promptly detected and rectified as necessary.

The brittle nature of concrete sometimes gives cause for concern in derailment damage. In practice, the enhanced stability of concrete-sleepered track reduces the incidence of derailments and their severity is reduced by modern fastenings which hold gauge well during derailments. Well-filled ballast cribs provide protection to the concrete and derailment damage to the track is not a significant problem with monoblock concrete sleepers. Twinblock sleepers, however, perform badly as the tie bar is relatively easily bent causing severe gauge narrowing.

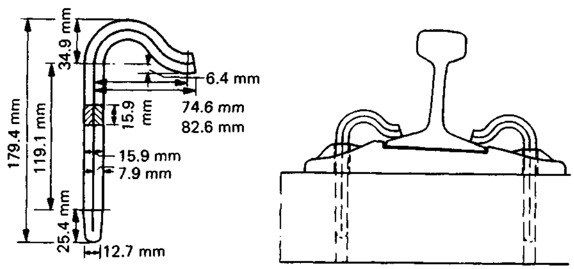
Whilst concrete is a semiconductor, adequate insulation for track circuits can be obtained simply and cheaply by incorporating insulating components into the fastening.

Concrete sleepers can be cast into a concrete slab to provide ballastless track but it is advisable to use sleepers with projecting reinforcement to ensure permanent connection to the *in situ* concrete. In dry locations, especially tunnels under sensitive buildings where structure-borne vibration is objectionable, attenuation can be achieved by enclosing the sleepers partially (without projecting reinforcement) in rubber boots before casting into the slab.

Pretensioned concrete beams have been in use in Britain as switch and crossing bearers since the early 1970s. Use was inhibited by the practice of drilling the concrete to locate rail fastenings, but manufacturers have now developed methods of casting fastenings into the bearers, and increasing use is anticipated internationally.

Figure 25.2 British Rail F27 prestressed concrete sleeper (*Courtesy*: Costain Concrete Co. Ltd)

Figure 25.3 Types of rail fastening: (a) spring spike, (b) Pandrol fastening, (c) K fastening, (d) square-section cut spikes, (e) FIST fastening



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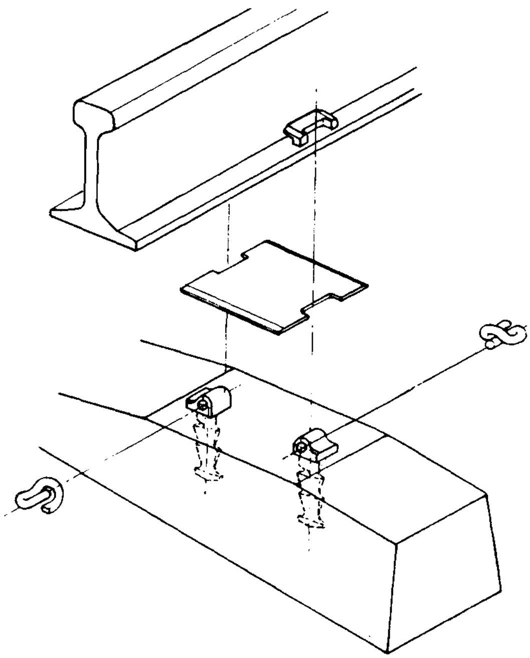

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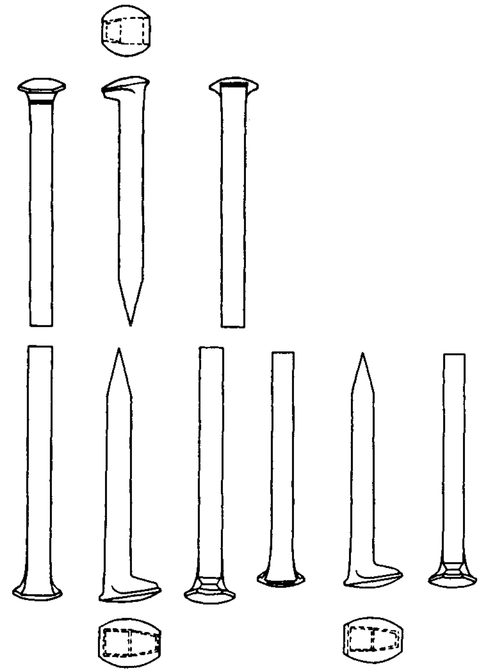

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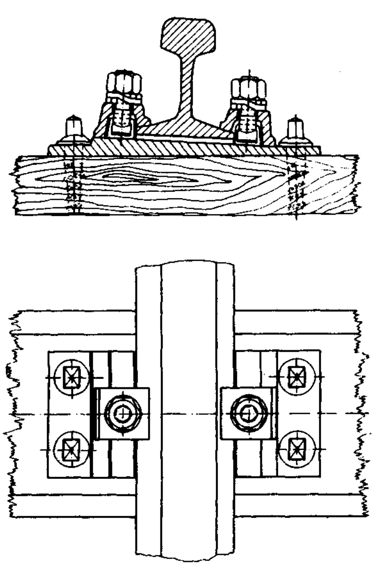

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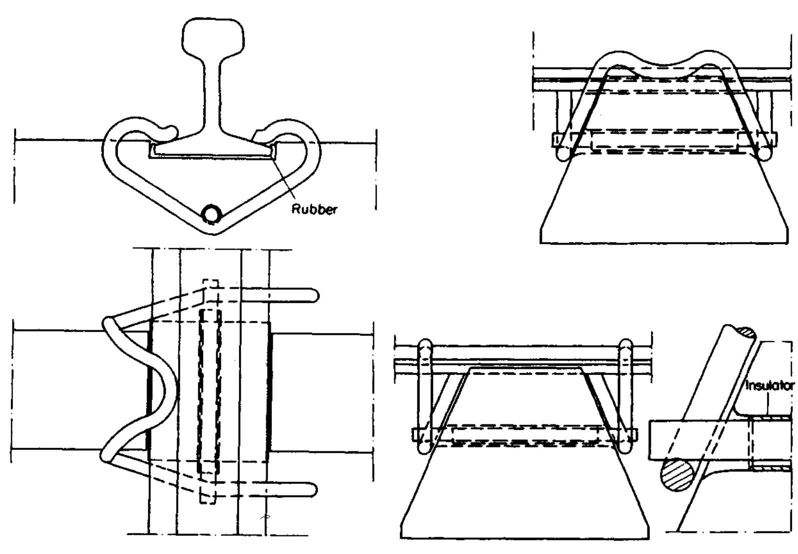


(e)

25.4 Fastenings

Rail fastenings are divided into two categories: direct and indirect. These are further divided into elastic and rigid. Direct fastenings are those which fasten the rail directly to the sleeper, and indirect are those which fasten the rail to a chair or baseplate, which is fastened to the sleeper separately.

Both types of fastenings may be either elastic or rigid, depending on whether a spring element is incorporated, and also depending on whether they are adjustable or self-tensioning.

The square-section cut spike was the original fastening used for flat bottom rail (Figure 25.3), but in Europe it is no longer used except in sidings and light, narrow-gauge railways. In North America, however, and on American-built railways in various parts of the world, it is still in general use on lines carrying axle loads of up to 35 t. Resilient forms of the cut spike have, however, been used extensively outside Canada and the US, particularly in Europe.

The cut spike has a number of disadvantages which have led to the decline in its use by European railways. The major disadvantages are: (1) short sleeper life due to spike killing and baseplate cutting; (2) the need for high sleepering density (in America timber sleeper spacing of 500 mm is still regarded as normal); (3) the need for rail anchors either side of the sleeper with heavy-section rail; and (4) variable track gauge. The cut spike is a simple and robust system which lent itself well to rapid pioneer developments, but it is poorly suited to high speeds and welded rail, especially in countries where temperatures vary between extremes of heat and cold. It is also of little use in countries where timber is not readily available.

In Europe, the coachscrew direct fastening has been preferred and is still used extensively. Both cut spike and coachscrew have poor gauge-holding when used as direct fastenings, but when used with baseplates, their performance is improved and this can be improved further when additional screws or spikes are used to secure the baseplate to the sleeper. It is now general practice to fit baseplates on all curved track fastened with spikes or screws.

The main advantage of the screw over the cut spike is that it exerts a positive pressure on the rail foot whereas the spike allows the rail to float 'free'. It therefore goes some way toward resisting rail creep and rail expansion due to temperature change. In France and on French-built railways, the generally accepted standard fastening is a combination of the screw spike used with a spring clip (called the RN fastening).

In Germany and much of Central Europe, the K fastening of the Deutsche Bundesbahn has been the standard fastening since 1926. It consists of a rolled steel baseplate giving a 1:40 rail inclination with two heavy ribs forming a channel seating for the rail resting on a 5 mm compressed poplar or plastic pad and slotted to take a T-headed bolt each side, inverted rigid U-shaped clips being held by the T bolts to which heavy spring washers are fitted. Four screws are used to hold the baseplate down to timber sleepers in Germany but, in some other countries, e.g. Belgium, only two are used. In Germany, spring steel washers are commonly used on baseplate screws. A number of spring clips have been developed which can be driven into the rib of the baseplate to replace the rigid clips.

In Britain, the Pandrol resilient rail fastening has been the standard British Rail fastening since 1964 on both concrete and timber sleepers, and also in S and C work. It is also used extensively in Australia, Africa, Canada, the Middle East and many other countries throughout the world. It has been adopted by the railways of sixty countries since its introduction, and is generally regarded as one of the simplest and most economical fastenings yet developed. Other resilient fastening systems which have been used quite widely are the DE, Heyback, SHC and FIST – the SHC in the UK and the FIST in South Africa

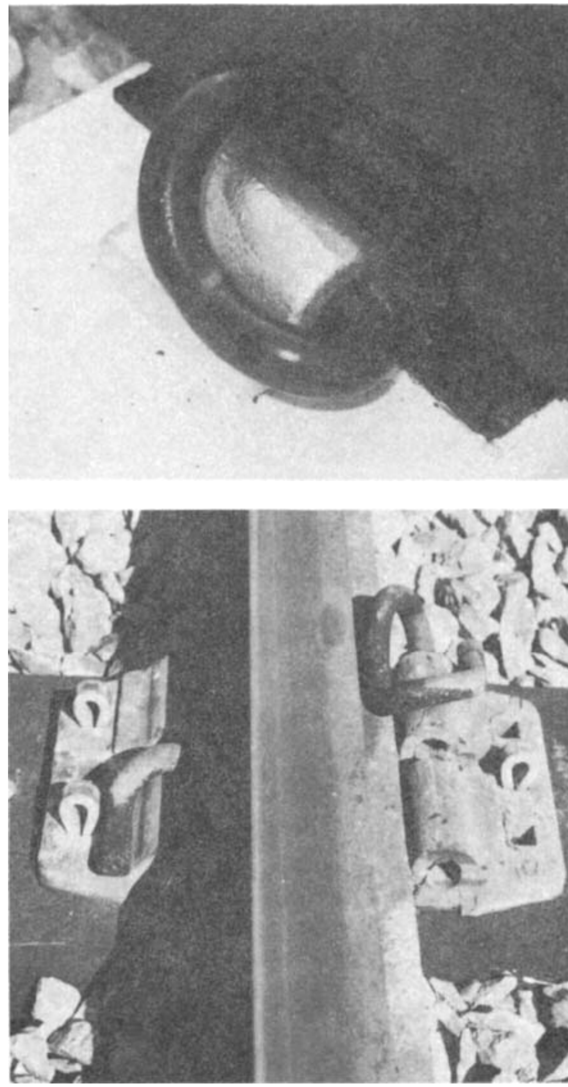


Figure 25.4 Pandrol clips on a concrete sleeper (top) and a timber sleeper (bottom)

and Sweden. In many parts of the world, the increasing shortage of track maintenance labour and its growing cost have led to greater interest in 'fit and forget'-type fastenings such as the Pandrol fastening, particularly for continuously welded rail. The importance of the fastening in welded-rail track being able to maintain a constant clamping force on the rail foot is fundamental to the stability of the track. It is of even more significance if the ballast condition is below standard.

The clamping force generated by the rail fastening must be able to resist any tendency for the rail to creep through the fastening, but not so much as to allow the rail to push the sleepers through the ballast section. A nominal clip pressure of between 1.4 and 2 t per rail is generally sufficient to achieve this.

It is important that any self-tensioning clip is designed to have a large deflection when fitted in position in order to minimize the effects of manufacturing tolerances on clips, rail pad, insulator, rail clip housing and rail foot. For example, a clip with 6 mm of nominal deflection will lose 50% of its clamping force with 3 mm of wear or tolerances, whereas with 12 mm of deflection, the toe load loss will be only 25%. The ability to ensure positive and accurate track gauge retention is also an important feature of a rail fastening.

Resilient rail fastenings are now used in the harshest of operating conditions, such as heavy haul and high-speed passenger services, and particular attention is being devoted to improving the ability of fastening assemblies to attenuate the transmission of dynamic forces from rail to sleeper. This is proving to be of particular importance in the case of fastenings used with concrete sleepers in high-speed passenger operations. Considerable effort is being focused on improving rail pad characteristics.

A further area of development in rail fastenings is the need for mechanical installation in those areas of the world where track labour costs are relatively high, and machines are now available to drive some of the more widely-used fastening types.

25.5 Rails

The flat bottom, or Vignole, rail is now an almost universal standard so that the obsolescent bull head rail of past British practice need not be developed here.

Generally, rail sections or weights are derived from progressive experience, not because of analytical difficulties but because of inadequate quantified knowledge of what conditions of loading and support actually occur.

A simple rough guide to appropriate rail weight in kilograms per metre is to multiply the axle load in tonnes by 2. However, it is necessary to make allowance for a speed factor and the following formula given by Schramm does this:

$$\text{Rail weight in kilograms per metre} = 156 - \frac{10\,600}{(A\alpha + 67)} \quad (25.1)$$

where A = static axle load in tonnes and α = the speed factor. The speed factor is still under critical review but it can be evaluated by three equations which are widely accepted for practical use:

$$\alpha = 1 + (V^2/30\,000) \quad \text{up to 100 km/h} \quad (25.2)$$

$$\alpha = 1 + (4.5V^2/10^5) - (1.5V^3/10^7) \quad \text{up to 140 km/h} \quad (25.3)$$

$$\alpha = 1.18 + (0.706V^3/10^7) \quad \text{over 140 km/h} \quad (25.4)$$

From the above expressions, Table 25.2 has been produced. The figures give an indication of appropriate weights within ± 15%.

British Standard specifications include rails from 25 to 56.5 kg/m; in Europe rails up to 60 kg/m are used and up to 70 kg/m in the US.

Table 25.2 Approximate rail weights, in kilograms per metre

Static axle weight (t)	*Speed* (km/h) 50	100	140	160	200
15	28	34	36	37	39
20	36	42	44	46	49
25	44	50	52	54	58
30	50	56	59	61	65
35	57	61	65	67	70

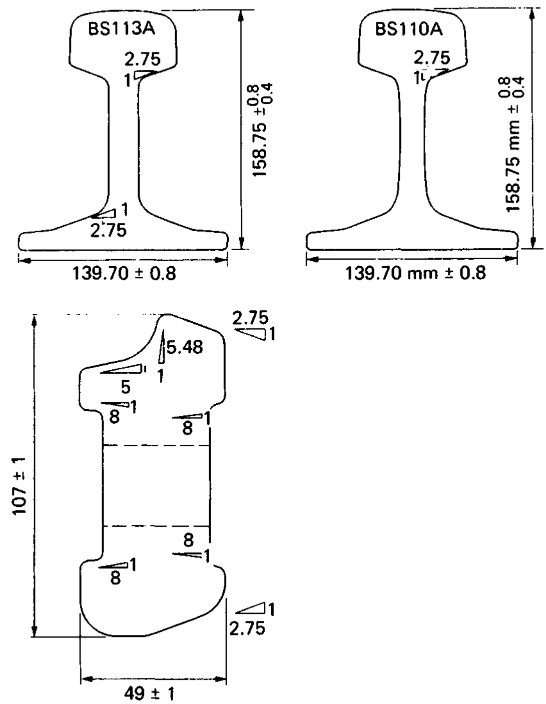


Figure 25.5 Sections of British Rail types 110A and 113A and fishplate (all dimensions in millimetres)

The present standard in Britain is the 56.5 kg/m (BS 113A) rail, adapted from the earlier British standard 109 lb/yd rail and its successor BS 110A section. Until relatively recently in Europe the standard section was the UIC 54 kg/m rail, but currently an increasing proportion of main line track is being constructed using UIC 60 kg/m section. In the USSR main line, heavy duty tracks incorporate 75 kg/m rail sections.

Table 25.2 relates to a representative sleeper spacing of 630 mm. If the spacing is wider than this, and until recently British practice utilized a sleeper spacing of 750 mm, the stress due to bending moment may be up to 9% greater, but in any case the occurrence of three loose sleepers can put up rail bending stress by 100% as can a heavy wheel flat at 30 km/h. At rail ends a 12 mm dipped joint can, with an unsprung mass of the order of 20% of the static load at a speed of 160 km/h, create a dynamic increment of load equal to the static load. Since the great majority of rail breaks occur at rail ends it can be stated that the rail weight selected must take into account the unsprung masses on the heavier axles and also the state of maintenance of joints which can be realized.

The weight of rail chosen has an influence on the shear loading of the subsoil. According to Eisenmann increasing the rail weight from 48 to 68 kg/m diminishes shear stress by 20%.

As rail weights increase, the breadth of the rail foot increases so that changing to a heavier-weight rail will generally involve changing the baseplates or, where cast-in inserts are used, respacing the inserts, unless the new rail is limited to new sleepers. Similarly, the administration of permanent way stocks is simplified, and the tied-up capital reduced, if the range of rail sections in use is limited.

The subsurface Hertzian stresses arising from wheel to rail contact must be very carefully considered, in view of their

Table 25.3

Country	*Max. axle load (t)*	*Rail section (kg/m)*	*Max. speed (km/h)*	*Rail steel grade used*	
				Straight track	*Curves*
Britain	26	56.4	200	Normal	Wear-resist.
Europe	22.5	60	200	Wear-resist.	Premium
US	28	68	120–130	Wear-resist.	Premium

relationship with wheel load, wheel size and tensile strength of the rail steel. The Hertzian contact stresses are directly related to the square root of the wheel load and inversely related to the square root of the wheel radius. The nominal shape of the rail head and wheel tread profiles and the progressive wear of these two profiles in service, also influence the actual wheel–rail contact area and, hence, the Hertzian contact stresses. The maximum level of the Hertzian contact stress normally occurs a few millimetres below the contacting surfaces, and shelling and similar subsurface fatigue failures can develop if the shear stress in this contact zone exceeds about 50% of the ultimate tensile strength of the rail steel. Under conditions of excessive stressing, complete collapse of the rail head can occur.

The principal rail specifications in international use are BS11 of the British Standards Institution, UIC 860-0 of the Union Internationale des Chemins de Fer (UIC) and the *Manual* of the American Railway Engineering Association (AREA). A norm for railway rails (ISO 5003) has also been published by the International Standards Organization (ISO).

The most commonly used rail steels are defined in the above standard specifications and can be classified as follows:

(1) Normal grades, typically of 680 N/mm^2 minimum UTS as per BS 11 normal grade and UIC 860-0 grade 70.
(2) Wear resisting grades, typically of 880 N/mm^2 minimum UTS as per BS 11 wear resisting grades A and B, UIC 860 grades 90A and 90B, AREA standard carbon grade.
(3) Premium high strength grade, typically of 1080 N/mm^2 minimum UTS.

The premium high-strength rail grades are still the subject of much research and development, and are briefly referred to in only one specification – AREA.

These premium grade rails are produced either by the use of alloy steels, generally about 1% chromium and in some cases with micro-alloying additions of vanadium, molybdenum or niobium, or by the heat treatment of wear-resisting grade rails, e.g. BS wear resisting grade A, UIC 860 grade 90A, or AREA standard carbon grade. The heat treatment may consist of either full section hardening or localized head hardening only.

The rail steel grade selected for a particular track is dependent on various factors including track design, traffic and operating conditions, standards of maintenance and economic factors, etc.

Table 25.3 illustrates the different optimum strategies selected by different railroads. It is common practice to utilize a higher-strength, more wear-resistant, rail steel grade in tight curves, and a less expensive grade in straight track.

Broad recommendations for the selection of rail steel grade based on track curvature and traffic intensity are outlined in UIC Code 721-R.

A special grade of rail steel utilized throughout the world, albeit in relatively small quantities, is austenitic manganese steel (12–16% Mn) which is usually used in heavily trafficked turnouts, switches and crossings. Many crossing vees (frogs) are produced as austenitic manganese castings. Rolled austenitic manganese rails are produced in Britain in relatively small quantities being used in the fabrication of turnouts and S and C work, and occasionally in curved track. Austenitic manganese steel surface hardens rapidly in service under stress and impact, and develops high wear resistance. However, it is expensive, needs great care in machining to avoid the risk of fatigue crack initiation and can only be welded to similar steel by normal rail welding techniques. Its higher thermal coefficient of linear expansion also produces manufacturing problems and service use limitations, but rolled austenitic manganese rails have been successfully welded and used in the relatively constant temperature conditions of underground railways.

Experimental work continues in the evaluation of more exotic and expensive rail steel chemistries for special applications, e.g. bainitic steels.

Rail joints are either suspended or supported. Joints are also square, i.e. opposite each other, or staggered by up to half the rail length. In some administrations, rail joints are square on the straight and staggered on the curves. Rail and fishplate sections need to be considered together in selecting a rail section, since if the ratio of the I_{xx} of the fishplate is much below 25% of the I_{xx} of the rail, broken fishplates and battered rail ends are likely to have a high incidence.

25.5.1 Rail failures

It is necessary in any railway system that a record should be kept of all cracked and broken rails removed from the track. In addition, the cause of failure must be noted in order to monitor any specific problems which may be developing. This is carried out with the use of a standard reporting form which gives details of track and conditions as well as the type of defect. This information is fed into a computer and frequent monitoring of the data gives any developing trends.

Whilst this latter information gives details of rails removed, it does not give any indication of those defects remaining in the track under observation. This information comes from the normal regular examination of the track which is carried out by means of an ultrasonic rail flaw detection train and also by hand-held ultrasonic rail flaw detectors used by pedestrian operators.

The ultrasonic test train is normally a self-propelled unit consisting of two vehicles. It operates at speeds up to 30 km/h and uses a series of probes which are applied to the rail head either in the form of sliding probes or wheel probes. The data from these probes are partially reduced by an on-board computer and the resulting information is stored on magnetic tape for subsequent off-line analysis. The defects detected by this system are sent to the appropriate maintenance engineers for action depending on the type of defect.

In addition to the train, hand-held ultrasonic units are used to carry out work in areas not covered by the test train and also to examine, in greater detail, defects which have been picked up by the test train but which require more detailed study. In addition, the hand units are also used for monitoring defects which are allowed to remain in track until such time as they can be repaired or removed.

Ultrasonic testing of rails is carried out at a frequency determined by the types and speeds of traffic carried. It ranges from 6 months to alternate years for some of the lightly used branch lines.

In addition to this routine testing, other specialized tests are carried out by both the test train and the hand operators which include gauge measurement, crack size estimation in special cases and weld testing.

25.6 Curved track

The main curves of the railway are nominally of constant radius, i.e. circular curves; curves made up of two or more circular curves of different radii curving in the same direction are called compound curves. Straights are generally and desirably connected to circular curves by transition curves of progressively varying radii; and adjoining circular curves of different radii are commonly joined together in a similar way if the difference of radii exceeds about 10%. Two adjoining circular curves curving in opposite directions comprise a reverse curve, and here, whatever the radius, the presence of connecting transitions is relatively more important than with circular or compound curves.

For practical purposes the cubic parabola $y = kx^3$ gives a uniform change of curvature between tangent point on the straight and tangent point on the curve, or between the tangent points of two curves, and is the most used form of transition curve. The versines, measured on half-chord overlapping chords along the transition, change with linear uniformity from zero to R although it is usual to smooth out the rate of increase and decrease at the start and end of the transition by putting on about one-sixth of the rate of the first versine at the zero station and reduce the increment at the final transition station to about five-sixths of the half-chord rate of increase. Versines are conveniently measured in millimetres.

The geometrical relation between a circular curve and a transition curve tangenting on to both the straight and the circular curve involves the moving inwards along a diameter normal to the straight of the circular curve by an amount termed the 'shift'. The transition curve bisects the shift at its midpoint measured along the straight tangent line and the offset from that line to the tangent point of the transition and circular curve is 4 times the shift or 8 times the offset at the mid-transition point. It follows that where no transition exists or where it is insufficient in length the institution or extension of a transition can be done only by sharpening the radius of the circular curve concerned or moving the tangent straight away from the curve, though this may be worthwhile since any transition is better than none.

The length of a transition curve is determined primarily by what is judged to be an acceptable rate of change of cant or cant deficiency. For standard gauge plain track a desirable rate may be 35 mm/s, with a maximum of, say, 55 mm/s to secure passenger comfort. In switches and crossings a rate as high as 80 mm/s may be applied but a good standard of switch and lead design is desirable for this rate of loss or gain of cant or cant deficiency.

Some limiting cant and cant deficiency values observed in British practice on a 1.432 m gauge are listed at top right.

Wherever space permits, curve design should be based on the desirable rate of change of cant or deficiency of 35 mm/s.

The amount of cant applied to a track depends upon consideration of the following factors:

(1) The line speed limit, i.e. the maximum speed at which traffic is allowed to run on a line or branch or section of a line or branch. This limit is usually fixed with reference to the value and distribution of permanent speed restrictions on the line or branch, or section thereof, involved.

Maximum cant:	
on curved track	150 mm
at station platforms	110 mm
maximum cant gradient	1 in 400
deficiency on plain line, CWR	110 mm
deficiency on plain line, jointed track	90 mm
deficiency on switches and crossings welded into CWR	
on through line	110 mm
on turnout	90 mm
with negative cant on turnout	90 mm
at switch toes	125 mm
deficiency on switches and crossings in jointed track	
on through line	90 mm
on turnout	90 mm
with negative cant on turnout	90 mm
at switch toes	125 mm
Maximum rate of change of cant	
on plain line	55 mm/s
on switches and crossings	55 mm/s
Maximum rate of change of cant deficiency	
on plain line	55 mm/s
on switches and crossings	55 mm/s
Maximum rate of change of cant deficiency	
on plain line	55 mm/s
on switches and crossings (inclined design)	55 mm/s
on switches and crossings (vertical design)	80 mm/s

(2) The proximity of permanent speed restrictions, junctions, stopping places, etc.
(3) Track gradients which may cause a reduction in the speed of freight or slow-moving passenger trains without having an appreciable effect on the speed of fast trains.
(4) The relative importance of the various types of traffic.

Generally, where fast and slow trains run on the same lines, an intermediate speed is selected to fix the cant. In this situation it may exceptionally be necessary to limit the cant and therefore the maximum speed to prevent surface damage to the low rail by heavy axles on slow-moving freight trains.

Each line of a double line should be separately assessed. In exposed locations subject to high winds it may be desirable to limit cant to below the 150 mm maximum.

Normally, cant or cant deficiency is uniformly gained or lost within the length of a transition curve or, where there is no transition curve, as may occur in switches and crossings, over the length of the virtual transition, which for practical purposes is taken as the shortest distance between the centres of bogies of coaching stock using the line. If the desired cant cannot be put on within this length observing a maximum cant gradient of 1 in 400, the cant loss or gain is continued on to the circular curve. The 1 in 400 cant gradient mentioned relates to axle twist derailment possibilities, especially where four-wheeled vehicles are concerned.

The maximum permissible speed on circular curves appropriate to the determination of permanent speed restrictions may, for standard gauge railways, be obtained from the following expressions:

$$\text{Equilibrium (or theoretical) cant} = E = 11.82(Ve^2/R)\ \text{mm}$$

$$\text{Equilibrium speed} = Ve = 0.29\sqrt{(RE)}\ \text{km/h}$$

$$\text{Maximum speed} = Vm = 0.29\sqrt{R(E+D)}\ \text{km/h}$$

where R = radius in metres, E = cant, which may be either actual cant or equilibrium cant but in practice the difference is not likely to be significant, though the distinction has to be kept in mind in certain circumstances, and D = maximum allowable cant deficiency in millimetres.

Desirable lengths of transition curves can be derived from the greater of the two values obtained from:

Length $= L = 0.0075 E Vm$ m
or $L = 0.0075 D Vm$ m

where E = cant in millimetres, D = deficiency of cant in millimetres and Vm = maximum permissible speed in kilometres per hour.

If space is limited, the length of the transition may be reduced to two-thirds L subject to a minimum cant or cant deficiency gradient of 1 in 400.

On compound curves to be traversed at a uniform speed, the desirable length is obtained from the greater of the two following expressions:

$L = 0.0075(E_1 - E_2)Vm$ m or
$L = 0.0075(D_1 - D_2)Vm$ m

where E_1 and D_1 are the cant and cant deficiency conditions for one curve and E_2 and D_2 are the similar values for the other curve.

Similarly, on reverse curves the transition lengths are given by:

$L = 0.0075(E_1 + E_2)Vm$ m or
$L = 0.0075(D_1 + D_2)Vm$ m

It should be noted that in using the above formulae the constants used have reference to a standard gauge, to an assumed height of the centre of gravity of vehicles of about 1.5 m and a subjective passenger comfort assessment of the tolerable rate of change of cant or cant deficiency. To this extent the values are arbitrary rather than absolute and refer to standard gauge. Further qualifications are that the springing of vehicles may be such as to produce excessive lean under cant deficiency running to encroach significantly on side clearances or to diminish passenger comfort, whilst if the centre of gravity of a vehicle is higher or lower than that assumed the maximum permissible speed is affected in inverse ratio, so that certain vehicles may be permitted to run at a higher speed than other vehicles.

The versine or middle ordinate of a chord on to a curve is proportional to the curvature and is the basis of all railway curve alignment, checking and adjustment. Its value as determined from triangular analysis is given by:

$$2R = \{(C/2)^2/V\} + V \qquad (25.5)$$

where R = radius of a curve, C = length of chord on which the versine is measured, and V = the versine, but since the value of V is very small in relation to R in railway situations, the final V of the expression may be disregarded so that for both field measurements and calculation purposes:

$$V = C^2/8R \qquad (25.6)$$

In Britain and on the Continent, railway curvature is usually described by the length of the radius measured in metres. The American practice is to describe a curve by the angle subtended at the centre of the curve by a chord of 30.48 m. For railway work it is sufficient in transposing degree units into radius units to divide 1746 by the degree of the curve to give the radius in metres. Thus, what is described in American practice as a 10° curve would be described in Europe as a curve of 175 m radius.

Main railway curves are initially set out by theodolite generally on the basis that equal chords are subtended by equal angles but informal setting out by an offset from a tangent followed by the use of overlapping chords of convenient length using the versine appropriate to the radius can give a degree of accuracy sufficient for less important curves. A selection of curve formulae covering most calculations for the design or setting out of railway circular curves is set out in Table 25.4.

It is desirable to keep rail joints more or less square, or at a constant stagger where this is preferred, on curves by inserting short rails in the inner rail of curved track. The difference in rail length, D, for standard gauge and for 18.3 m rails is given by D = (27.45/radius in metres) m. Alternatively, the approximate difference in length can be obtained from the formula $D = \frac{3}{4}$ versine. It is normal to take a standard range of short rails from the makers and for 18.3 m rails, these are 18.25, 18.20 and 18.15 m.

The precise alignment of curves is vital to a smooth ride, stability of the track geometry and minimizing track wear. There are several alternative methods of adjusting railway curves as a maintenance operation but all are based on versine measurements and relate, either to a smoothing or averaging approach, in which the differences between a limited number of adjacent versines are adjusted to give a fairly uniform rate of change in transitions or to a fairly uniform value on circular curves; or to a design lining approach in which the whole of the versines of a curve are adjusted as one revision operation to give a precise rate of increase or decrease in transitions and a strictly uniform value on the circular curve portion, as far as clearances from structures, etc. will permit. It is generally convenient to use smoothing or local adjustment techniques after a curve has been aligned on a design basis. It is also normal practice to set markers or monuments or pegs in or beside the tracks when design lining is carried out.

Curve realignment can be based on manual measurement of versines or with less accuracy on the versine measurements of a track geometry recording car or of a track lining machine.

The side and interbody clearances shown on the diagram contained in the Department's Requirements (see Figure 25.15) relate to straight track and may need to be augmented on curved track for end throw or centre throw of vehicles, which are generally of the same magnitude.

Centre throw $= C = B^2/8R$

End throw $= E = (L^2 - B^2)/8R$

where B = wheelbase or bogie centres, L = length of vehicle, and R = radius.

To this increase of effective body width on curved track must be added a further allowance due to the tilting of the vehicle to the low side on canted track. In Britain, the loss of side clearance at the vehicle cornice is about 2.25 times the actual cant.

It is general practice slightly to widen the track on very sharp curves to allow all vehicles, especially locomotives with long rigid wheelbases, to pass round such curves without straining the track. The normal British practice is:

Curves over 200 m radius widened to 1435 gauge
200 to 110 m radius widened to 1439 gauge
110 to 70 m radius widened to 1451 gauge

Appropriate values of gauge widening for a given track radius depend on free play of wheelsets, length of half wheel flange below rail level and rigid wheelbase of the critical vehicle.

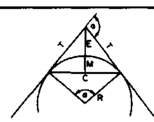
In the UK it is the Department of Transport's enjoined practice to install check rails on curves of 200 m or less radius.

Table 25.4 Curve formulae

Given	*Sought*	*Formula*	*Given*	*Sought*	*Formula*
E, M	C	$C=2M\surd\{(E+M)/(E-M)\}$	T, α	M	$M=T\cot(\alpha/2)\,\text{vers}\,(\alpha/2)$
E, R	C	$C=2R\surd\{E(2R+E)\}/(R+E)$	D	R	$R=50/\sin(D/2)$
E, T	C	$C=2T(T^2-E^2)/(T^2+E^2)$	C, E	R	$0=R^3+R^2\{(4E^2-C^2)/8E\}-(RC^2/4)-(C^2E/8)$
E, α	C	$C=2E\{\sin(\alpha/2)\}/\{\text{ex sec}\,(\alpha/2)\}$	C, M	R	$R=\{(M^2+(C/2)^2\}/2M$
M, T	C	$0=C^3-2TC^2+4M^2C+8M^2T$	C, α	R	$R=C/2\sin(\alpha/2)$
M, R	C	$C=2\surd\{M(2R-M)\}$	E, α	R	$R=E/\text{ex sec}\,(\alpha/2)$
M, α	C	$C=2M\cot(\alpha/4)$	M, E	R	$R=EM/(E-M)$
R, T	C	$C=2TR/\surd(T^2+R^2)$	M, α	R	$R=M/\text{vers}\,(\alpha/2)$
R, α	C	$C=2R\sin(\alpha/2)$	T, C	R	$R=CT/\surd\{(2T+C)(2T-C)\}$
T, α	C	$C=2T\cos(\alpha/2)$	T, E	R	$R=\{(T+E)(T-E)\}/2E$
R	D	$\sin(D/2)=50/R$	T, M	R	$0=R^3-R^2(M^2+T^2)/2M+RT^2-MT^2/2$
α, L	D	$D=100\alpha/L$ approx.	T, α	R	$R=T\cot(\alpha/2)$
C, M	E	$E=M(C^2+4M^2)/(C^2-4M^2)$	C, E	T	$0=2T^3-T^2C-2TE^2-CE^2$
C, α	E	$E=C\{\text{ex sec}\,(\alpha/2)\}/2\sin(\alpha/2)$	C, M	T	$T=C(C^2+4M^2)/2(C^2-4M^2)$
M, α	E	$E=M/\cos(\alpha/2)$	C, α	T	$T=C/\{2\cos(\alpha/2)\}$
R, C	E	$E=R^2/\surd\{(R+C/2)(R-C/2)\}$	E, α	T	$T=E\cot(\alpha/4)$
R, M	E	$E=RM/(R-M)$	M, E	T	$T=E\surd\{(E+M)/(E-M)\}$
R, T	E	$E=\surd(T^2+R^2)-R$	M, α	T	$T=M\{\tan(\alpha/2)\}/\{\text{vers}\,(\alpha/2)\}$
R, α	E	$E=R\,\text{ex sec}\,(\alpha/2)$	R, C	T	$T=CR/2\surd\{(R+C/2)(R-C/2)\}$
T, C	E	$E=T\surd\{(2T-C)/(2T+C)\}$	R, E	T	$T=\surd\{E(2R+E)\}$
T, M	E	$0=E^3+E^2M-ET^2+MT^2$	R, M	T	$T=R\surd\{M(2R-M)\}/(R-M)$
T, α	E	$E=T\tan(\alpha/4)$	R, α	T	$T=R\tan(\alpha/2)$
R, α	E	$E=R\{1-\cos(\alpha/2)/\cos(\alpha/2)$	D, L	α	$\alpha=DL/100$ approx.
α, D	L	$L=100\alpha/D$ approx.	M, C	α	$\tan(\alpha/4)=2M/C$
C, E	M	$0=M^3+M^2E+(MC^2/4)-(C^2E/4)$	M, E	α	$\cos(\alpha/2)=M/E$
C, α	M	$M=(C/2)\tan(\alpha/4)$	R, C	α	$\sin(\alpha/2)=C/2R$
E, α	M	$M=E\cos(\alpha/2)$	R, E	α	$\text{ex sec}\,(\alpha/2)=E/R$
R, C	M	$M=R-\surd\{(R+C/2)(R-C/2)\}$	R, M	α	$\text{vers}\,(\alpha/2)=M/R$
R, E	M	$M=RE/(R+E)$	R, T	α	$\tan(\alpha/2)=T/R$
R, T	M	$M=R-\{R^2/\surd(T^2+R^2)\}$	T, C	α	$\cos(\alpha/2)=C/2T$
R, α	M	$M=R\,\text{vers}\,(\alpha/2)$	T, E	α	$\tan(\alpha/4)=E/T$
T, C	M	$M=(C/2)\surd\{(2T-C)/(2T+C)\}$	R, α	L	$L=0.017\,453\,292\,5R\alpha$
T, E	M	$M=E(T^2-E^2)/(T^2+E^2)$			

D = Degree of curve
R = Radius
α = Ext. angle = central angle
L = Length of curve

M = Mid-ordinate
T = Tangent
C = Long chord
E = External distance



Note: ex sec A = sec $A - 1 = (1 - \cos A)/\cos A$ *and* vers $A = 1 - \cos A$

Check rails are also sometimes installed in Britain on flatter curves to diminish side cutting of the high rail or as a protection against inadvertance or mishap at the bottom of a gradient. Wide flangeway check rails or guard rails may also be installed to protect bridge supports or girders against being struck by derailed vehicles. In the UK this provision is a Department of Transport Requirement.

25.7 Welded track

About two-thirds of the on-track maintenance work put into jointed permanent way is at or adjoining the rail joints and the great majority of rail failures occur at or near fishplated joints. For these and other reasons the tendency today is to use long welded or continuously welded rails.

This procedure entails the elastic containment of the longitudinal expansion and contraction stresses arising in the rails as a consequence of variation in the rail temperature with reference to the temperature at which the rails were fastened to the sleepers. If, as is usual, the rails are fastened down on installation within a prescribed narrow temperature range situated at or a little above the mean of the annual rail temperature range in a normal or stress-free condition, then the amount of tension in winter and compression in summer is limited to what is judged to give an optimum compromise between a cold season hazard of broken rails and a hot season hazard of track buckles.

The installation and maintenance of welded rail track need more technical insight and attention than ordinary track. The lateral strength of the track depends upon three main elements: (1) the I_{yy} of the two rails; (2) the framework stiffness of the assembled rails and sleepers as developed by the fastenings; and (3) the frictional loading of the ballast on the sides, ends and bottoms of the sleepers. The resistance of the track to vertical buckling, which generally occurs in combination with, or as a trigger to, lateral buckling, is determined mainly by the total weight of the track, and in this context concrete through-type sleepers show a considerable advantage. In a fully-ballasted track the ballast representatively accounts for about two-thirds of the total moment of lateral resistance. Loose sleepers and kinks in the alignment are of especial significance to the stability of welded track so that a high standard of maintenance is essential for this sort of permanent way.

25.7.1 Fastening welded rails

The need to fasten the rails in order to get a stress-free rail at about the mean of the annual temperature range would mean, in many parts of the world, a very short season for laying welded rails without artificial assistance. This assistance is normally supplied either by heating the rails by propane gas travelling heaters or by using hydraulic tensors, so that when fastened down after extension by heat or tensile pull, the rail has a length equivalent to a stress-free condition at a temperature between prescribed limits (21 to 27°C for the UK). In this way it is possible to lay welded track all the year round. The pull required to extend a 56 kg/m rail is of the order of 1.6 t/°C. The tensors have a capacity of about 70 t.

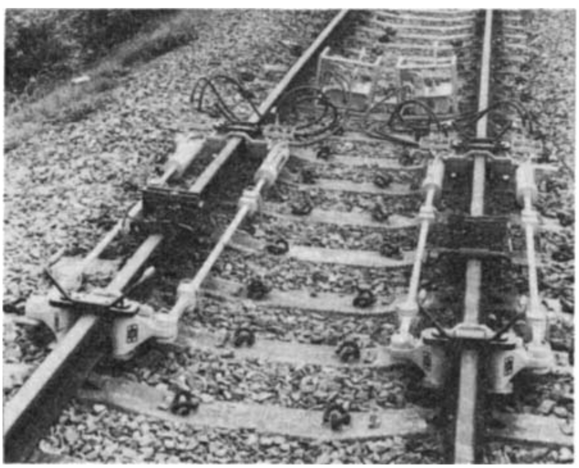


Figure 25.6 Hydraulic rail stresser. This has a 70-t pull and 380-mm stroke. For pushing action, the rail clamps are turned through 180°. (*Courtesy*: Permaquip Co. Ltd)

Welded rails are not normally laid in Europe in curves with a lesser radius than 600 m. Long welded rails are made up by flash butt welding of standard rail lengths into welded rails of 200 to 400 m length in depot and welded into continuous welded rails by Thermit welds, except in the USSR where a transportable flash butt welder is applied to join rails in the track.

It is normal practice in Britain and France to install sliding switches at the ends of long welded rails but in Germany this is not done, reliance being placed on very firm fastening of the welded rails where they connect with jointed track.

The advantages of welded track include an extension of rail lives by about a third, a reduction of on-track maintenance by about half, a dramatic reduction of rail breakages, an increase in running speeds, less damage to the formation, increased sleeper lives, improved ride comfort, a reduction of traction energy of up to 5%, and a reduction in train noise. In effect, welded rails, prestressed concrete sleepers and self-tensioning fastenings are complementary to each other in extending the life and reducing the long-term overall cost of permanent way.

25.8 Switches and crossings

All railway switch and crossing work is built up of three basic units: (1) switches; (2) common or acute (angle) crossings; and (3) obtuse (angle) or diamond crossings. Layouts of switches and crossings are illustrated in Figures 25.7 to 25.10.

25.8.1 Switches

The heel of a switch is the point from which it is free to move. Early switch designs had a loose heel formed by a semi-tight fishplated joint but this type is now rarely used outside of sidings. It is now the general practice for the heel of the switch to be formed by a bolted connection through a block between switchrail and stockrail so that the switch joint is well behind the heel. The length of the switch planing on the bead of the rail varies between about 1.6 and 11 m and most railway administrations have a range of two to eight standard switches to meet the requirements of short leads in depots and various speeds of traffic on the running lines.

Originally, ordinary or straight planing of switch blades was universally applied but during the last few decades curved planing, in which the switch rail is bent elastically whilst being planed on one side of the head, has become general. In this way the curve running through the lead is continued to the switch tip. A recent practice is to apply a larger radius to the planing of the switch than is applied to the switch beyond the end of the planing to give a more robust switch tip with a finite entry angle.

25.8.2 Crossings

Crossings are described by the angle at the crossing nose. In continental Europe it is common to state this in degrees but over the English-speaking world the angle is usually described as 1 in *N*. However, the angle of a crossing expressed as 1 in *N* varies slightly according to the measuring convention adopted. Thus if 1 in 8 is obtained by measuring 8 units along the centreline for 1 unit of symmetrical spread normal to the centreline (centreline measure), then measuring 8 units along one leg of the vee for a spread of 1 unit measured at right angles to that leg (right angle measure) will give 1 in 7.969, whilst if 8 units is measured along each leg to give a spread of 1 unit the crossing size will be 1 in 8.016 (isosceles measure). It is therefore necessary for any railway to have a specific mode of measuring crossings which is known to all supplying manufacturers of crossings, or to state the angle in degrees.

In the UK fixed diamond crossings are limited by Ministry requirement to be not flatter than 1 in 7.5. In continental Europe 1 in 10 is the general limit. Where a line crossing flatter than the allowed limit is necessary, the points of the diamond crossings are constructed as switches and have to be set to suit the train movement desired.

There is no official limit on the angle of common crossings but 1 in 28 is about the flattest angle at which the crossing nose has sufficient robustness. Crossings are either made up from rails or are cast in 12 to 14% manganese steel. Built-up crossings have the inherent limitations of bolted assemblies, and of recent years it has become general practice in the UK to weld the vee section and Huck-fasten the wings. In general, it is preferred practice to have cast crossings for high-speed or heavily worked lines using built-up crossings for less exacting duty. It is also preferred to use cast crossings for angles flatter than about 1 in 20. In the US it is common practice for the nose section of the crossing to be a manganese casting to which ordinary rail section wings are attached to form what is known as a rail bound cast crossing.