

LONG WELDED RAILS

INDIAN RAILWAYS INSTITUTE OF CIVIL ENGINEERING
PUNE

PREFACE

Long Welded Rail (LWR) has now become synonymous with modern track structure with a major portion of Indian Railways track having long welded rails. It is imperative that permanent way men understand all its facets , be it welding, laying or maintenance so that full benefits are reaped . With this objective, IRICEN publication on LWR was printed in 1988 which of course requires revision. This publication is an updated version with a completely new look incorporating the latest correction slips and provisions of the LWR Manual.

The publication highlights the evolution of the LWR over the years with brief references to the research work carried out in RDSO and foreign railways on various aspects of the LWR. A brief description of the various SEJ layouts now available, latest provision of LWR on bridges with comments on the state of art, neutral temperature and its measurement are also included. It is hoped that this publication will go a long way in helping track engineer to understand the intricacies involved in laying and maintaining LWR track.

This book has been authored by Shri Ajit Pandit, Sr. Professor & Dean of this Institute. If there are any suggestions or discrepancies, kindly write to the undersigned.

Shiv Kumar
Director
IRICEN

ACKNOWLEDGEMENTS

While covering the subject on Long Welded Rails at IRICEN the absence of an updated publication on the subject covering the state of art and latest instructions was acutely felt. The publication printed in 1988 required revision to incorporate the provisions of the LWR Manual 1996, including the latest correction slips.

This IRICEN publication is a result of the desire to fill the gap and produce a documentation which would be useful for all practicing civil engineers on Indian Railways.

It would be appropriate to mention the support and assistance rendered by IRICEN faculty and staff in preparing this publication. Special mention may be made of Shri Sunil Pophale, Head Draftsman who rendered valuable assistance in preparing the drawings. Shri Dhumal, PA assisted in editing the manuscript. Shri R.K. Verma, Senior Professor/Track gave valuable suggestions from time to time.

Above all, the author is grateful to Shri Shiv Kumar, Director/IRICEN for his encouragement and guidance for preparing the document.

Ajit Pandit
Senior Professor & Dean

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CHAPTER - I

INTRODUCTION TO LONG WELDED RAILS

1.1 Evolution of Long Welded Rails

1.1.1 The dream of a jointless track has fascinated track engineers ever since the first railway was laid and efforts are continuing even today for achieving a fully continuous track without joints. To achieve this various innovative techniques have been used over the years. Two ways to achieve this are:

(1) Increasing the length of rails by rolling longer rails.

(2) Welding the rail joints.

1.1.2 At the beginning of the last century the standard length of rail was generally 12m. Apart from the logistic considerations of rail transportation and its loading and unloading, the length of the rail was governed by the length of cooling boxes in the rail manufacturing steel plant, as controlled cooling after the rolling process was necessary. Subsequent advancements in the manufacturing process have enabled rolling of longer rails possible. 26m long rails are now being manufactured by SAIL in the Bhilai Steel Plant. Rails of 65m and 78m and even longer rails upto 480m are being planned for manufacture in the near future in the steel plants across the country. Jindal Steel & Power Ltd are setting up a steel plant in Raigad dist. of Chhattisgarh state, where longer length rails are likely to be manufactured.

1.1.3 Although welding of rails was started in 1905 itself, commercial welding on any considerable scale became common only after 1932. During the thirties,

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the weights and lengths of standard rail sections varied from 22kg/m to 65 kg/m and from 5.5 m to 27m respectively (Reference1). The length of welded panels varied from 18m to 380m. The welding process most commonly used at that time was 'thermit' though flash butt and other processes were in use in America, South Africa and elsewhere.

- 1.1.4 Although longer length rails were being used, track engineers were unsure of the expansions which these longer length rails would undergo under temperature variations and consequently, the expansion gaps required at the ends of these panels. Studies by the British Transport Commission (Reference 2) however, indicated that the *gap to be provided is not proportional to the length of the welded rail* and in fact the expansion gaps are actually independent of the length of the rail. What this implies is that in long welded panels, free rail expansion or contraction is not being permitted and thereby undergo what are called as 'thermal stresses' due to constrained expansion and contraction. The ability of the welded panel to withstand compressive stress buildup due to constrained expansion was another source of worry to the early track engineers. The British Transport Commission conducted a number of studies in the 1950s to go into the aspect of lateral stability of the Long Welded Rails under temperature induced compressive stresses. Interestingly it was found that these welded panels derive resistance to buckling not only from the stiffness of the rail itself but also from the rail sleeper fastenings and ballast resistance. It was only after the lateral stability of rails under compressive forces was confirmed was the concept of Long Welded Rails accepted by track engineers.

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1.1.5 In India in the nineteen thirties, the GIP Railway had undertaken welding of rail joints using the electrical process. In 1939, BN Railway started conducting trials on welded rail joints. Between 1940 and 1946, other railways viz. NW, GIP and EI railways also commenced trials with welded joints. From 1947 to 1966 large number of 5-rail panels (65m in BG and 60m in MG) and 10-rail panels (130m in BG and 120M in MG) were put into track. The purpose was to reduce the maintenance efforts by reducing the number of joints. However, large scale maintenance problems were reported by various railways regarding the behaviour of 5-rail and 10-rail panels due to:

- i) increased rail battering and hogging;
- ii) elongated fish-bolt holes;
- iii) bent fish-bolts.

The phenomena of battering and hogging of rail joints is shown in Fig. 1.1

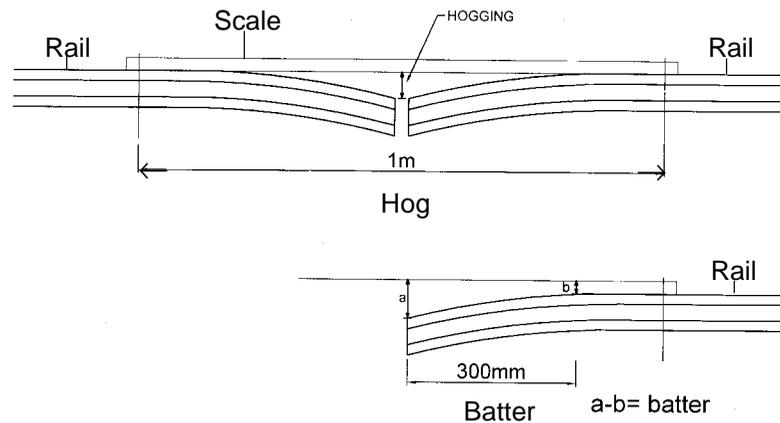
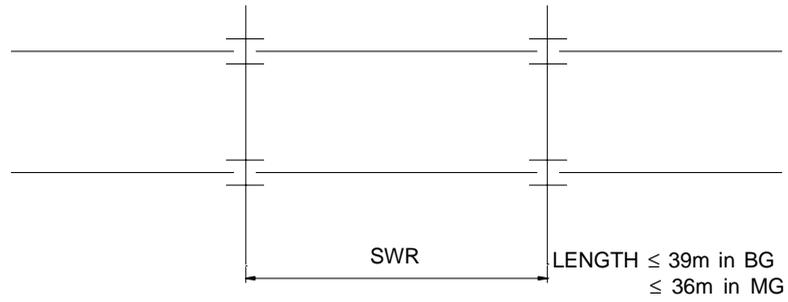
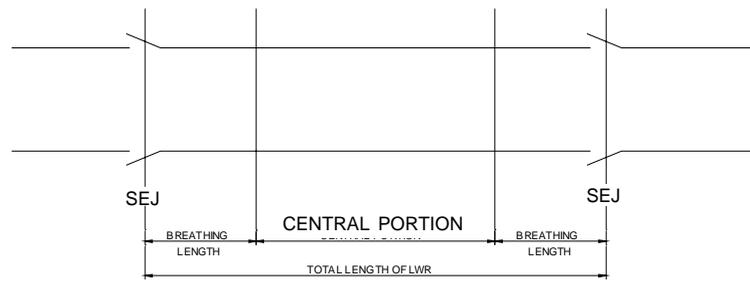


Fig. 1.1

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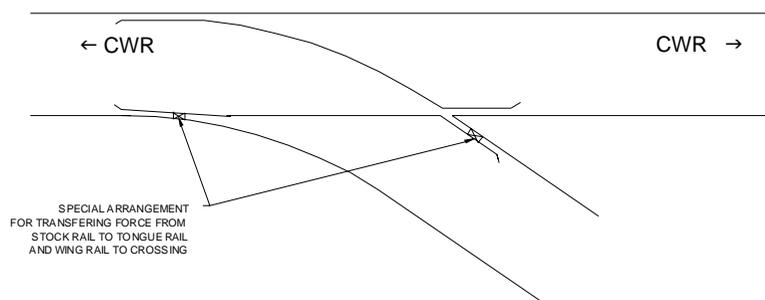


SWR
Fig. 1.2(a)



Minimum length of LWR
= 250m in BG
= 500m in MG
MAX. LENGTH : ONE BLOCK SECTION LENGTH

LWR
Fig. 1.2(b)



CWR
Fig. 1.2(c)

(5)

Taking cognizance of these problems, the Railway Board vide their letter No 65/W6/WRL/6 of 20th January 1966 appointed a committee consisting of 3 Engineers in Chief (Track), Joint Director Standards (Track), RDSO and a Deputy Chief Engineer of W.Rly to investigate into the behaviour of 5-rail panels and 10-rail panels at the first instance and thereafter of Long Welded Rails.

1.1.6 The findings and recommendations of the committee make interesting reading (Reference1). The committee found that the IRS fishplate design as per current standards is inadequate to cater to the expansion and contraction occurring in 5-rail and 10-rail panels. While the capacity of the IRS fishplate design is to accommodate a movement of 15 mm, the actual movements of a 5 rail or 10 rail panel are much larger resulting in large gaps, bent fishbolts and elongated fishbolt holes. The 3-rail panel therefore appears to be roughly the longest rail which could be laid in the track with the conventional fish plated joints. The committee, therefore, recommended that:

- 1) Welding of 5-rail and 10-rail panels was to be discontinued,
- 2) Existing 5-rail and 10-rail panels were to be cut into two and half rail panels;
- 3) RDSO was to conduct further studies for deciding the track structure, temperature and ballast conditions for laying the LWR.

1.2 Some Basic Definitions

- 1) SWR : A rail which expands and contracts throughout its length: 3-rail panels, 39m in BG and 36m in MG, will behave like an SWR.(Fig1.2(a))
- 2) LWR : (LWR) is defined as a welded rail whose central portion does not exhibit any

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longitudinal movement on account of temperature variations. The movements are exhibited by track lengths on either side of the central portion which are called as 'breathing lengths'. While the maximum length of an LWR is restricted to one block section length, the minimum length of an LWR should be 500m in MG and 250m in BG. (Fig 1.2(b))

- 3) CWR : An LWR which continues through station yards including points and crossings is described as a continuous welded rail. (Fig 1.2(c))
- 4) Rail Temperature : The temperature of the rail as measured by an approved rail temperature measurement device.
- 5) Breathing length : The lengths at the ends of an LWR on either side where the LWR "breathes" i.e. exhibits movements due to temperature variations is called the breathing length.
- 6) Destressing Temperature (t_d): The average rail temperature at the time of fixing the rail fastenings after a destressing operation performed manually is called the destressing temperature.
- 7) Stress-free Temperature or Neutral Temperature (t_n) : The rail temperature at which an LWR is free of longitudinal thermal stress is called the stress-free or neutral temperature.
- 8) Switch Expansion Joint (SEJ) : A physical device placed at the end of the breathing length of an LWR to accommodate the expansion and contraction of the breathing length.
- 9) Buffer rails : A set of free rails used in lieu of SEJs as a temporary measure to accommodate the movement of the breathing length of the LWR.

1.3 An Explanatory Note on the Short Welded Rail

Indian Railways have adopted a 3-rail panel (39m in BG and 36 m in MG) as the Short Welded Rail(SWR) with fish plated joints due to reasons mentioned in 1.1.6. Although the SWR is exhibiting expansion and contraction throughout its length, it is not a free expansion or contraction due to the action of ballast and fastenings. The SWR therefore develops thermal forces because of being subjected to constrained expansions and contractions. It has been estimated that the magnitude of thermal force in an SWR may be as high as 70% to 75% of the thermal force in an LWR. This coupled with fish plated joints makes the SWR very vulnerable and should therefore be maintained with care. The provisions of laying and maintenance of SWRs is given in para 505 of the Indian Railways P-way Manual (1986).

1.4 Advantages of the Long Welded Rail

Today the LWR is synonymous with modern track. The reason for the widespread popularity of the LWR are the numerous advantages which are derived using a LWR track structure. The LWR makes train travel more safe, economical and comfortable due to the reasons listed below :

- LWR tracks eliminate fishplated joints leading to safety. Sabotage at fish plated joints has been a major worry for the Indian Railways and this has again come into prominence with the accident of the Rajdhani Express at Rafiqganj in Bihar in which a large number of people lost their lives and which was attributed to removal of fishplates on the approach of a bridge.
- Fish plated joints are a source of large dynamic forces. As a result fish plated joints exhibit large scale rail wear and development of cracks from fishbolt holes and fractures. In some instances premature rail renewal may have to be carried out due to excessive fractures.
- Due to development of large dynamic forces at the rail joints the track geometry at the rail joint gets disturbed

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frequently resulting in an increment in the track maintenance effort. It has been estimated that there is as much as 25% to 33% savings in the track repair and maintenance costs due to elimination of rail joints.

- Due to impact at rail joints there is an added wear and tear of rolling stock wheels to an extent of 5% and as the wheel has to negotiate the gap there is added fuel consumption to an extent of 7%.
- Due to elimination of noise and vibrations at the rail joints passenger comfort is substantially increased.

CHAPTER II

PRINCIPLES OF LONG WELDED RAIL

2.1 Basic Principles

A metal rod supported on frictionless rollers can theoretically expand and contract freely with variations in temperature. If the length of the metal rod is 'L', it will undergo a change in length equal to $L \alpha t$, where α is the coefficient of linear expansion and t is the change in temperature.

For example, if there is a steel rod 13 m long (the same as the standard length of a rail), then it will undergo change in length equal to 3 mm, if the temperature of the rod changes by 20°C as can be seen from the calculation below:

$$\text{Expansion/Contraction} = L \alpha t = (13 \times 1000 \text{ mm}) \times (1.152 \times 10^{-5}) \times 20 \\ = 3 \text{ mm.}$$

($\alpha = 1.152 \times 10^{-5}$ per degree centigrade)

However, the rail in the track cannot be compared to the metal rod supported on frictionless rollers as mentioned in the preceding paragraph. The rail is restrained from free expansion and contraction by the sleepers because of,

1. Creep resistance on account of friction between the rail and sleeper at the rail seat.
2. Creep resistance further offered by the rail-sleeper fastening.

Thus the expansion or contraction of the rail is less than what it would undergo if it was completely unrestrained.

In LWR the rail is held down to the sleeper by fastenings which have adequate toe load, thereby preventing any relative movement between rail and sleeper. Thus with any change in temperature it is not the rail alone but the rail-sleeper frame as a whole which tends to move.

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Here again the rail sleeper frame is not entirely left unrestrained. The frame is under restraint because of the resistance offered by the ballast in which the sleepers are embedded. The resistance offered by the ballast to the movement of the track frame in the direction of the track is called longitudinal ballast resistance. Longitudinal ballast resistance is assumed to be uniform for sleepers of the same type. This longitudinal ballast resistance builds up progressively from the ends of the long welded rail towards the centre.

If the track frame was not restrained, then the rail would expand or contract with variation in temperature and consequently no force would build up in the rail. However, since there is a restraint now offered by the longitudinal ballast resistance, forces are set up in the rail. These are called thermal forces. If the temperature variation (from the temperature at which the rail was fastened to the sleeper) is small, then a small length of track at the end would be sufficient to develop longitudinal ballast resistance against the tendency for free movement of the rail. Consequently, it would lead to a small amount of thermal force in the rails. However, If the temperature difference becomes more, a longer length of track at the ends would be called upon to develop the necessary longitudinal ballast resistance against the free movement of the ends of the rails and there would be a corresponding increase in the thermal forces.

There is however, a limit up to which the temperature differences can build up. This limit is dictated by:

- a) either the maximum or the minimum rail temperature and
- b) the temperature at which the rail was fastened to the sleeper.

The temperature at which the track could be attended under regular track maintenance, the temperature at which the patrolling of the track should be introduced, etc. are governed by the sole consideration that the thermal forces at any time in the track should be within safe limits, to avoid the eventualities of buckle and fracture of the LWR.

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2.2 Force Diagram

In the previous paragraph it has been indicated that thermal forces are built up in the LWR due to the resistance offered by the ballast to the free expansion or contraction of the track frame. An expression for the magnitude of this thermal force is required as this will govern the design of the LWR. Let us assume the LWR central portion to undergo a change of temperature equal t °C temperature rise. If the central portion of length 'L' had been free to expand it would have expanded by an amount equal to ' $L\alpha t$ '. However since the central portion of the LWR does not move, the compressive strain induced in central portion is equal to

$$\frac{L\alpha t}{L} = \alpha t \quad \text{where 't' is the change of temperature of the LWR}$$

with respect to the temp. at which the LWR was laid or distressed and α is the coefficient of linear expansion.

If P is the force induced in central portion (compressive force) and A is the cross section area of rail, then P/A is the compressive stress in the rail.

Since stress /strain = E (Young's modulus of rail steel)

we have
$$\frac{P/A}{\alpha t} = E$$

or $P = A E \alpha t$ where P is in Newtons, A is in mm^2 , and E is in N/mm^2 .

As t represents the change of temperature of the rail w.r.t. the temperature it is stress free, a technically correct formula for the thermal force in the LWR will be

$$P = A E \alpha (t_p - t_n)$$

where t_p is the prevailing rail temperature and t_n is the rail neutral temperature, which is the temperature at which the LWR is free of longitudinal thermal stress.

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When $t_p = t_n$, $P=0$,
when $t_p > t_n$, P is a compressive force
and when $t_p < t_n$, P is a tensile force.
As t_n is not known it is assumed that $t_n = t_L$ or t_d , temperatures at which LWR was laid (t_L) or destressed (t_d).

The force at the beginning of the LWR is zero, and in the central portion equal to $A E \alpha t$. This change of force from zero to a peak value occurs over the breathing length. The shape of the force diagram is therefore as given in Figure 2.1

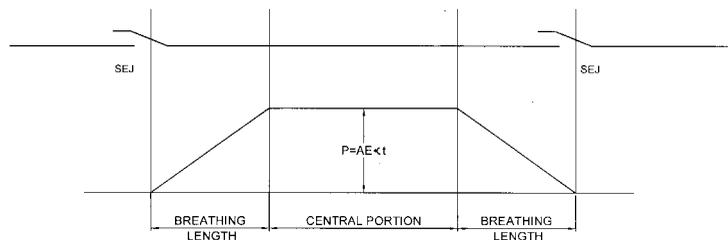


Fig. 2.1

Thermal stress in rail due to unit degree change of temperature
 $= E \times \alpha = 2.15 \times 10^6 \times 1.152 \times 10^{-5} = 2.5 \text{ MPa/}^\circ\text{C}$
For 30°C change of temperature therefore thermal stress induced
will be about 75 MPa which is about 8.5% of the ultimate tensile
strength of a 90 UTS rail.

2.3 Importance of Rail Temperature

From the expression for force in an LWR, $P = AE\alpha(t_p - t_n)$, it can be seen that force developed in the LWR depends primarily on the prevailing rail temperature and the rail neutral temperature. Both these factors are discussed below:

2.3.1 Rail Temperature Measurement :

Thermometers

The following are the different types of approved thermometers for measuring rail temperature :

- i) Embedded type - This is an ordinary thermometer inserted in a cavity formed in a piece of rail-head, the cavity filled with mercury and sealed. The rail piece is mounted on a wooden board which is placed on the cess and exposed to the same conditions as the rail inside the track. This type of thermometer takes 25 to 30 minutes for attaining temperature of the rail.
- ii) Dial type - This is a bi-metallic type thermometer which is provided with a magnet for attaching it to the rail. The thermometer is attached on the shady side of the rail web as this location is approximating the average rail temperature to the greatest extent. A steady recording of the rail temperature is reached within 8 minutes.
- iii) Continuous recording type - It consists of a graduated chart mounted on a disc which gets rotated by a winding mechanism at a constant speed to complete one revolution in 24 hours or 7 days as applicable, giving a continuous record of rail temperature. The sensing element is attached to the web of the rail and connected to the recording pen, through a capillary tube which is filled with mercury.
- iv) Any other type of thermometer approved by RDSO/Chief Engineer.

Where a number of thermometers are used to measure the rail temperature at one place, as in case of laying of LWR, distressing etc. any of the thermometers showing erratic readings, appreciably different from the other adjoining thermometers, shall be considered as defective.

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Zonal Railways should nominate 8 to 10 stations in their railway in a manner as to give the representative sample of the temperature variations on the Zonal Railway for the region allotted to each station. These stations shall be the existing PWIs offices. On these stations rail temperature records shall be built up using preferably a well calibrated continuous recording type thermometer. The maximum and minimum rail temperature for a continuous period of at least 5 years shall be ascertained and the mean rail temperature (t_m) for the region arrived at. These temperature records shall be analysed to assess the probable availability of time periods during different seasons of the year for track maintenance, destressing operations and requirements of hot/ cold weather patrolling etc. Rail thermometers shall also be available with each gang and sectional PWIs to enable the gangs to work within the prescribed temperature ranges.

2.3.2 Rail Temperature Zones and RDSO Studies :

In order to understand the correlation between the rail temperature and ambient temperature, RDSO conducted rail temperature studies between 1969 and 1971 over a two year period. 22 stations were identified over the Indian Railways where Standard Measuring Arrangements for Rail Temp (SMART) were set up. (Fig. 2.2) SMART consisted of a full-length rail laid in the east-west direction on wooden sleepers with ACB plates and boxed with standard ballast profile. The rail temperature was measured by means of a thermometer placed in a mercury-filled hole in the rail head. Rail temperature readings were taken on an hourly basis between August 1969 and August 1971, and the corresponding air temperatures obtained from the Meteorological office. Correlation equations between the rail temp and air temp were derived using a computer based regression analysis (Details available in RDSO/C/146, Reference-3) for each of the identified 22 stations. Using these correlation equations and the maximum and minimum air temperatures at 180 stations over the Indian Railways, obtained from the Weather office over a period of 90 years, it was possible to determine the maximum and minimum rail temperatures obtainable at these stations. This data was presented in the form of a rail

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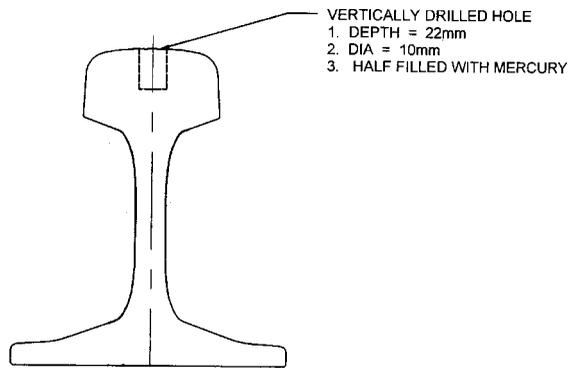
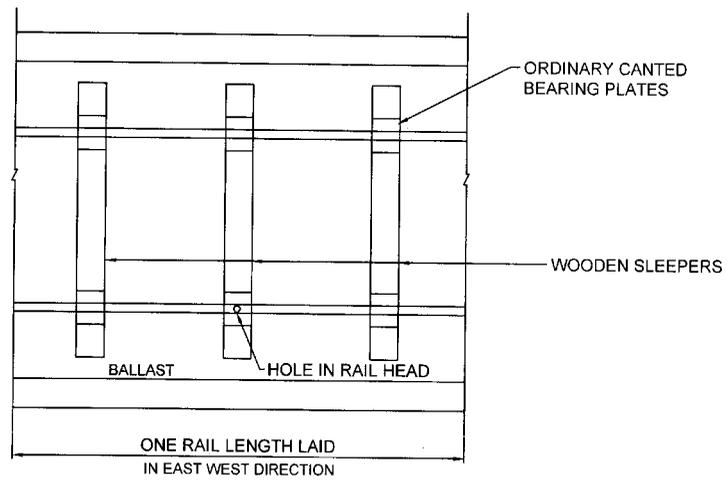


Fig. 2.2 : STANDARD MEASURING ARRANGEMENT FOR RAIL TEMPERATURE.

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temperature map where against each station two figures are given, namely the rail temperature **range** and the rail temperature **mean**.

The Rail Temperature Range = Max Rail Temp - Min Rail Temp

and Mean Rail Temperature = $\frac{\text{Max Rail Temp} + \text{Min Rail Temp}}{2}$

The country is further divided into four zones depending upon the temp range :

Temp. Zone	Temp. Range
I	40-50°C
II	51-60°C
III	61-70°C
IV	71-76°C

The rail temperature map is given in the LWR Manual (1996).

2.3.3 Rationale behind choice of t_n or t_d

The LWR neutral temperature should be chosen in such a manner that the thermal force developed in the LWR is within the desired limits.

Refer to Fig 2.3. The block shows the maximum, minimum and range of rail temperature in the four temperature zones. The rail can be fixed to the sleepers by fastenings after destressing, at a temperature anywhere within the range between maximum and minimum rail temperatures.

Let us see what happens if we fasten the rail to the sleeper at the minimum rail temperature (t_{min}). As the rail temperature rises compressive thermal forces will be built up and when the rail temperature reaches t_{max} compressive forces proportional to the full range of rail temperature will be built up. Such large compressive forces could be very dangerous to the stability of the LWR and the track can buckle. In this case there is of course no danger of any tensile force developing in the rail and consequently of rail fracture.

(17)

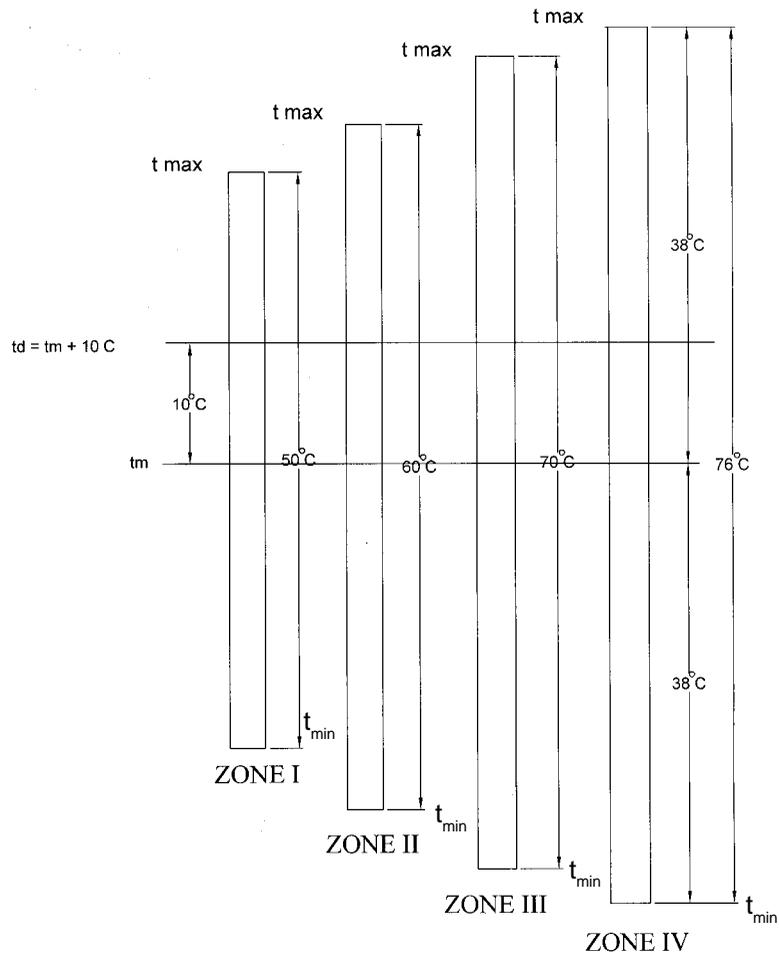


Fig. 2.3 : VARIATION OF TEMPERATURE WITH RESPECT TO t_d IN DIFFERENT ZONES.

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Let us see what happens if the rail is fastened to the sleeper at the maximum rail temperature (t_{max}). Since temperature cannot rise any further, there is no likelihood of compressive thermal stresses developing in the rail, and consequently there is no danger of buckling. However, since the rail temperature can fall through the complete range of temperature at this place, tensile stresses and forces in the rail could develop to a very large magnitude making rail fracture very probable.

Logic suggests that we should fix the destressing temperature exactly mid-way between maximum and minimum rail temperatures. i.e. at mean rail temperature. In that case the extent of maximum compressive or maximum tensile forces would be equal and half of what it would otherwise be as in case of either of the two previous situations. All the same compressive forces and tensile forces are bound to develop and so the possibility of buckling as well as fracture will exist. A fracture will create a gap in the rail. However, in the case of fractures, the alignment of the rail is not immediately distorted. Also fractures rarely occur on both rails and at the same location simultaneously. Thus at least a few trains can pass over a fractured rail without accident till the fracture is attended to. However, if the track buckles due to excessive compressive forces in the rail, alignment of track gets distorted and safe running of trains is endangered.

Therefore considering buckling more dangerous, it is considered prudent to fix the destressing temperature higher than the mean rail temperature so that the compressive forces built up in the track would be within reasonable limits, though at the cost of introducing higher tensile forces. This is the basis for fixing t_d on the Indian Railways at a temperature above t_m . Since 90R and lighter sections do not have adequate margin to accommodate the resulting thermal tensile stresses, t_d for such rails has been fixed between t_m and $t_m + 5^\circ \text{C}$. Heavier 52 kg and 60 kg rails having greater section modulus can accommodate relatively larger thermal tensile stresses and so t_d for these rails has been fixed between $t_m + 5$ and $t_m + 10^\circ \text{C}$. It also gives relief by reducing the compressive forces which are directly proportional to area of cross section of the rail, other conditions remaining the same.

(19)

Since the operation of fastening of the rails to the sleepers after destressing takes time during which rail temperature can vary, a range has been recommended for t_d instead of a fixed value.

In addition to the above consideration of avoiding buckling while risking the chances of fracture there are some more reasons why the destressing temperature has been fixed at a level higher than t_m .

- 1) Studies on LWR behaviour have indicated that the stability of the track gets endangered at temperatures above $t_d + 10^0$ C. Hence the clause of the LWR Manual limiting the maintenance operations on the LWR to a rail temp of $t_d + 10^0$ C has been provided. If the destressing temperature of the rail is lowered the prevailing rail temperature could rise beyond $t_d + 10^0$ C more frequently reducing the availability of maintenance hours. This would either entail limited working hours to the gangs, night working or working in split shifts. These options have serious practicability problems. Hence the destressing temperature of the LWR has been fixed at a temperature **above** the mean temp so that the prevailing rail temp does not frequently go above $t_d + 10^0$ C, restricting the maintenance operations, and ensuring adequate maintenance hours.
- 2) The other provision of the manual is that hot weather patrolling should be introduced when t_p rises above $t_d + 20^0$ C. Again if t_d is set at a lower temp. as compared to t_m , the prevailing rail temperature could rise above $t_d + 20^0$ C quite often leading to an increase in the quantum of hot weather patrolling.
- 3) The LWR manual prescribes imposition of speed restrictions at locations where track maintenance activities have been carried out and the temperature rises beyond $t_d + 20^0$ C during the period of consolidation. If t_d is fixed at a lower level, this would necessitate imposition of a large number of

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restrictions at work sites, whenever the rail temperature rises beyond $t_d + 20^\circ\text{C}$ during the period of consolidation. This situation is unacceptable.

Keeping the above factors in mind t_d has been fixed at a higher level as compared to t_m . The policy makers at that time knew that doing this will have an effect on the number of fractures and therefore indicated that a greater frequency of USFD inspection and monitoring would be required. Fractures are today one of our major concerns and there is a demand from some railways to reduce the level at which the destressing temperature should be fixed basically to reduce the number of fractures. A more realistic view would be to have a flexible neutral temperature which could be shifted downwards in winter to control fractures and shifted upwards in summer to control the tendency of the track to buckle.

2.4 Breathing Length

2.4.1 As discussed earlier the force build up in the LWR starts in the breathing length at the free end. The tendency of the rail sleeper assembly to expand or contract is resisted by the resistance of the ballast called the longitudinal ballast resistance. It is denoted by R and its unit of measurement is in kg/metre/rail. The longitudinal ballast resistance is mobilized when there is a relative movement between the sleeper and the ballast. If LB is breathing length and R the longitudinal ballast resistance, then $LB \times R$ represents the total resistance offered by the ballast in the breathing length.

As the maximum force in the LWR = $AE \alpha t$

$$LB \times R = AE \alpha t \quad \text{or} \quad LB = \frac{AE\alpha t}{R}$$

The above expression for the breathing length indicates the various factors which govern the breathing length as under:-

- 1) The breathing length is proportional to the temperature

(21)

change. Therefore the breathing length is maximum in Zone IV and minimum in Zone I.

- 2) The larger the cross sectional area of the rail, the larger the breathing length.
- 3) The larger the value of 'R' the longitudinal ballast resistance, the smaller the breathing length. As BG sleepers have a larger value of R the breathing length of BG LWRs is smaller as compared to MG LWRs.

Annexures 1A, 1B & 1C of LWR Manual give indicative values of breathing lengths for different sleepers and sleeper densities in different zones.

2.5 Longitudinal Ballast Resistance

The longitudinal ballast resistance 'R' comes into play when there is relative motion of the sleepers with respect to the ballast in the longitudinal direction.

2.5.1 RDSO conducted a number of experimental studies on the various aspects of the longitudinal ballast resistance. These studies are described in RDSO Report No. C-148(Reference 4).

Experimental Set up : These studies were conducted experimentally on a running track as well as on a freshly laid track in the lab. A track section comprising of short length rails and three sleepers embedded in ballast was pushed in the longitudinal direction and the displacement versus load curve plotted. The tests were conducted on a running line where a traffic block of 90 minutes was taken. The load was applied to this test panel by a hydraulic jack with a remote controlled pumping unit. The load applied was measured with the help of a proving ring and the longitudinal movement of the panel was recorded with the help of dial gauges. The instantaneous loads and movements were measured as the load was increased gradually till it reached a peak and fairly steady value. After the test, the short length rails were replaced by the normal rails. (Refer to figure 2.4)

The maximum value obtained from the proving ring was divided by twice the number of sleepers in action to get the ballast resistance

(22)

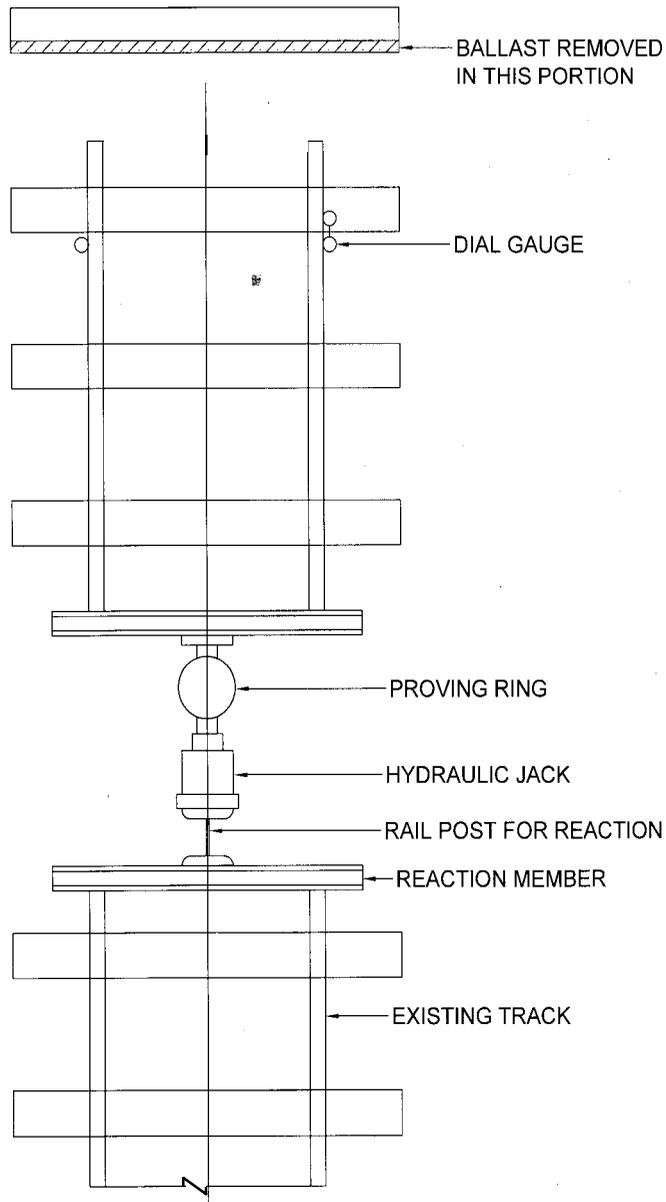


Fig 2.4

(23)

per sleeper per rail. This figure divided by the sleeper spacing in metres gives the value of 'R' in kg/m/rail.

Findings of the study :

1. In BG, PRC sleepers give the highest longitudinal ballast resistance in all conditions. In consolidated and through packed conditions other sleepers in order of decreasing longitudinal resistance are: Steel, RCC twin block, CST-9 and Wooden. In deep screened or freshly laid track, the order is PRC, RCC, Steel, Wooden and CST-9.
2. Through packing causes a reduction in ballast resistance. For both BG & MG concrete sleepers the average reduction is 23%. For BG conventional sleepers the reduction is 36%.
3. Deep screening causes a greater reduction in ballast resistance.
4. The effect of traffic on the growth of the ballast resistance is substantial. BG concrete sleepers attain 86% of the consolidated value on passage of 1 GMT of traffic.
5. Ballast resistance per sleeper decreases as the sleeper spacing reduces, but the ballast resistance per unit length of track remains more or less constant for sleeper densities from 1200 nos. to 1500 nos. per km. For larger sleeper densities the value of the longitudinal ballast resistance again increases due to heavier track structure.
6. A heaped up shoulder ballast gives a higher ballast resistance as compared to the standard shoulder for both BG and MG. This increase is maximum for concrete sleepers.

A summary of values obtained for different sleepers under different conditions is given in Table1, Table2 and Table3.

(24)

Table 1

**Longitudinal Ballast Resistance (kg/m/rail)
(Effect of sleeper type and track maintenance activities)**

Gauge	Sleeper Type	Conso- lidated	Through packed	% Loss	Deep Screened	% Loss
BG	PRC	1244	1027	17	885	29
	Steel	1051	744	29	433	59
	CST-9	933	551	41	276	71
	RCC	921	666	28	581	37
	Wooden	697	380*/552	45*/21	370	47
MG	Wooden	405	180*/302	55*/21	259	36
	Steel	298	231	22	209	30

* Values are for tracks packed by MSP
Conventional sleepers are beater packed
PRC sleepers have been off track tamped.
Sleeper spacing PRC – 1600 Nos/km
RCC and conventional – 1200 Nos/km
Adapted from RDSO/C-148

(25)

Table 2

**Maximum Longitudinal Ballast Resistance (kg/m/rail)
in freshly laid conditions
(Effect of sleeper density & heaped up ballast profile)**

Ballast	38 mm Standard Ballast profile				38 mm Heaped up Ballast profile			
	A	B	C	D	A	B	C	D
PRC	546	559	561	628	—	—	648	—
Steel	347	375	387	434	—	—	438	—

A : 1200 Nos per km B : 1400 Nos per km
C : 1600 Nos per km D : 1800 Nos per km

Adapted from RDSO/C-148

Table 3

**Longitudinal Ballast Resistance (kg/m/rail)
(Effect of movement of traffic)**

Sleeper	consolidated Value	Resistance on passage of 1 GMT of traffic	% of consolidated Value
PRC	1244	1072	86
Steel	1051	740	70
CST- 9	933	580	62
RCC	921	750	81

Adapted from RDSO/C-148

2.6 Lateral Ballast Resistance

The lateral ballast resistance comes into play when the track has a tendency to get displaced in the lateral direction due to build up of compressive forces. RDSO studies(Reference 5) conducted on various aspects of lateral ballast resistance have indicated the values of lateral ballast resistance as given in the Table below. The test setup is given in Fig. 2.5.

Values of Lateral Ballast Resistance in Kg/m of track

Gauge : BG Sleeper	Conso- lidated	Through Packed	Deep Screened
CST-9	1640	1100	532
PRC	1470	1226	1040
Steel	1430	825	540
RCC	1420	1040	800
Wooden	1060	320/520	384

Adapted from RDSO/C-156

1. The higher resistance recorded by metal sleepers is due to the central keel in the case of CST-9 sleepers and turned down ends in case of steel sleepers which get embedded in the ballast core and offer better resistance to the lateral movement.
2. The reduction in the lateral ballast resistance on through packing and deep screening is quite significant for CST-9 and steel sleepers and much less for PRC sleepers.

Tests conducted have revealed that :

- ◆ Track surfacing and ballast tamping even with a minor amount of rail lift (0.5 to 1 inch) can cause significant reduction in lateral track strength.

(27)

- ◆ Depending upon the ballast type, recovery of strength loss due to traffic could vary from 0.3 GMT to 9 GMT.
- ◆ Dynamic track stabilizers could significantly accelerate ballast consolidation or strength recovery. For instance, for granite ballast, the dynamic track stabilizer may produce a consolidation equivalent to 0.3 GMT.

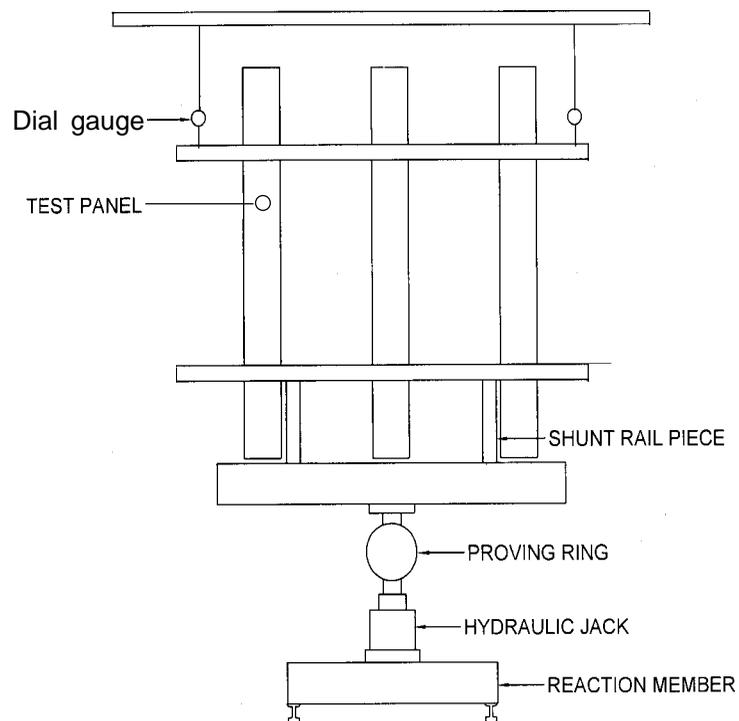


Fig. 2.5

CHAPTER III

THERMAL MOVEMENTS AND HYSTERESIS

3.1 Estimation of Thermal Movements

It is only in the breathing lengths that a LWR displays the property of longitudinal movements. At the ends of the LWR since the restraint offered by the longitudinal ballast resistance is nil, the movement is observed to be the maximum. As the longitudinal ballast resistance exerted on the sleepers progressively builds up complimentary forces in the rail increase from A towards B. (Fig.3.1) At B, which is the junction between the breathing and fixed lengths, the movement reduces to zero. The movements recorded in the field at various points in the breathing length of the LWR corroborate the above mentioned observations. It is possible to make certain simplifying assumptions and estimate the movement at any point in the breathing length.

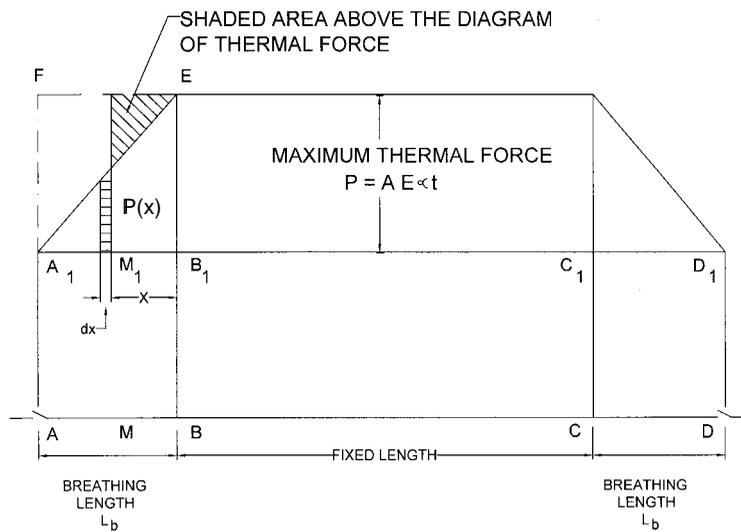


Fig. 3.1

(29)

Take a small length of rail dx at any arbitrary point M at a distance 'x' away from B (refer fig. 3.1).

It is possible to calculate the amount of free expansion of the small rail of length 'dx' due to change of rail temperature as well as the amount of contraction due to presence of thermal force present in the rail at that length:

Free expansion of this small length dx due to a rise in rail temperature by $t^\circ\text{C} = dx\alpha t$. The amount of contraction of this length dx , is equal to $\frac{P(x) dx}{EA}$

(where $P(x)$ is the thermal force present in the small length of rail dx at a distance x away from B).

The net expansion of the small length of rail dx will therefore be the difference between the above two values. If this net expansion is called dy , then

$$\begin{aligned} dy &= \alpha t dx - \frac{P(x) dx}{AE} \\ &= \frac{AE\alpha t dx - P(x) dx}{AE} = \left[\frac{P - P(x)}{AE} \right] dx \end{aligned}$$

where $P = AE \alpha t$ is the maximum thermal force in the central portion of the LWR.

Integrating the net expansion of all such small lengths of rails starting from B towards M, we can obtain the total expansion or displacement of points M as

$$Y = \int_0^x dy = \frac{1}{AE} \int_0^x [P - P(x)] dx$$

It can be observed from Fig. 3.1 that the expression $(P - P(x))dx$ is nothing but the area of the shaded diagram appearing above the diagram of thermal force.

Thus the amount of maximum contraction or expansion at any point

(30)

in the breathing length of a LWR can be computed by dividing the shaded area from B, upto that point as in the Fig. 3.1 by EA. Extending this logic, the cumulative value of expansion or contraction at the end of the LWR i.e. at 'A' or 'D' can be obtained as follows:

Maximum expansion or contraction at 'A' or 'D' =

$$\frac{\text{Area of Triangle } A_1FE}{AE}$$

$$= 1/2 \times P(LB)/AE \dots (\text{Equation 1})$$

As $P = AE \propto t$, equation (1) above can be simplified as maximum movement at the end of a LWR, m

$$= ((LB) \propto t)/2 \dots (\text{Equation 2})$$

We can look at it this way :

The maximum movement of the end of the LWR is half the corresponding value if only the breathing length LB of the LWR is allowed to expand or contract absolutely freely. The variation of movement along the breathing length is given in Fig.3.2.

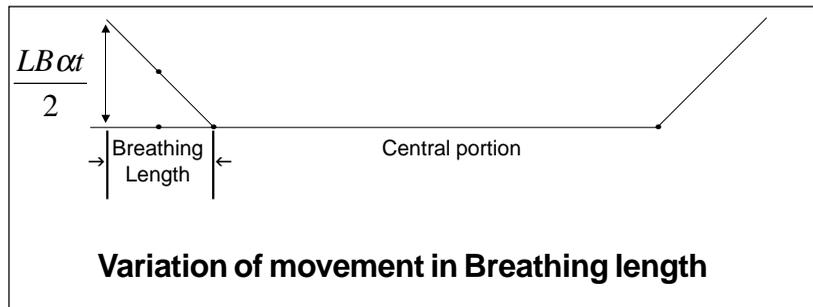


Fig. 3.2

This expression for the maximum movement at the end of the LWR i.e. $m = (LB \propto t)/2$ can also be rewritten as:

(31)

$$m = AE (\alpha t)^2 / 2R \dots \text{(Equation 3)}$$

This can be derived from Equation 1 by substituting $AE\alpha t/R$ in place of LB because

$$LB = P/R = AE \alpha t/R$$

It should be noted that in the above calculations an important assumption has been made that in a breathing length of LWR, the sleepers have equal values of longitudinal resistance.

To illustrate the above let us solve an example with the data given below:

Gauge: BG, Sleepers: PRC, Rails: 52kg ($A=66.15\text{cm}^2$)

Sleeper density 1540 sleepers/km, Zone IV with temp. range =76°C

$R = 13.28 \text{ kg/cm/rail}$

$A = 66.15 \text{ cm}^2$, $E = 2.15 \times 10^6 \text{ kg/cm}^2$

$$t_d = t_m + 10^0 \text{ C}$$

$t = 28^\circ\text{C}$ for temp. rise

$t = 48^\circ\text{C}$ for temp. fall

$$\alpha = 1.152 \times 10^{-5} /^\circ\text{C}$$

As discussed earlier movement at an SEJ $\Delta = \frac{AE(\alpha t)^2}{2R}$

For a temperature rise of 28°C,

$$\Delta_1 = \frac{66.15 \times 2.15 \times 10^6 (1.152 \times 10^{-5} \times 28)^2}{2 \times 13.28}$$

= 5.57 mm (forward movement of tip of tongue/stock rail wrt reference line)

(32)

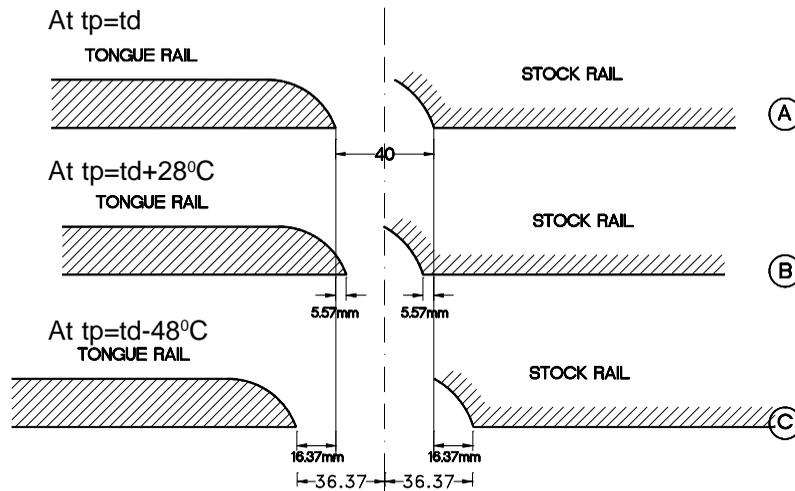


Fig. 3.3

For a temperature fall of 48°C

$$\Delta_2 = \frac{66.15 \times 2.15 \times 10^6 (1.152 \times 10^{-5} \times 48)^2}{2 \times 13.28}$$

= 16.37 mm (backward movement of tip of tongue/stock rail wrt reference line)

Total movement at the SEJ joint

$$= 2 \times (5.57 + 16.37)$$

$$= 2 \times 21.94$$

$$= 43.88 \text{ mm}$$

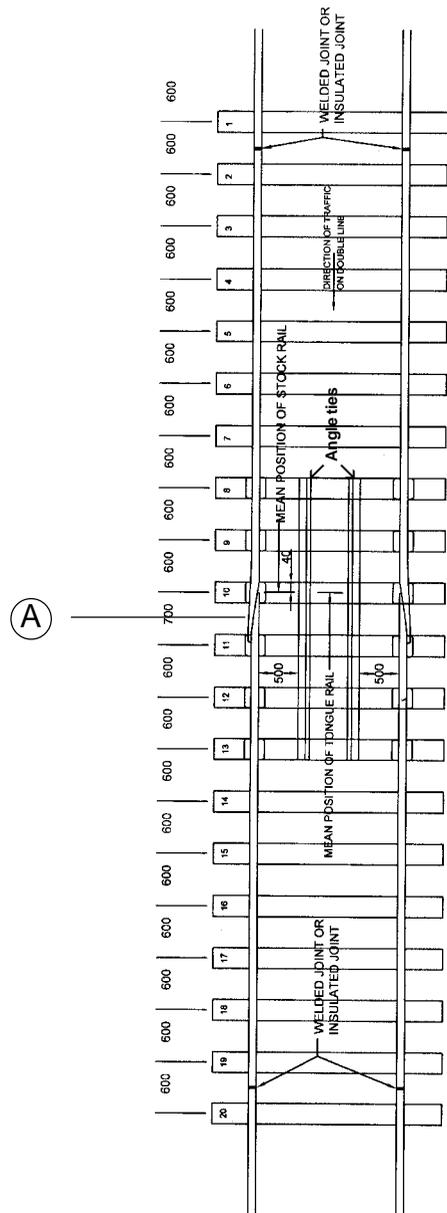
The movements which occur at the SEJ due to thermal variations are shown in Fig. 3.3 (A,B,C) with values as calculated in the above example.

3.2 Switch Expansion Joints (SEJ) :

The thermal movement in the breathing length of an LWR are accommodated at the switch expansion joint (SEJ). An SEJ typically consists of a pair of tongue rails and stock rails, the tongue rail laid facing the direction of traffic. Modern SEJs are laid on concrete sleepers with rail free fastenings. The tongue rails and stock rails are machined and given suitable bends to accommodate each other. Tongue rails and stock rails have typically straight alignment and hence these SEJs cannot be laid in curves sharper than 0.5° . The distance between the tip of the tongue rail and notch of the stock rail is typically kept as 40mm at the destressing temperature. Earlier the gap used to be 60mm and has now been reduced to 40mm to lower the impact of the passing wheel. Various types of SEJs used on Indian Railways are described below:

Drawing No.	Description
RDSO/T-4160	Assembly for Switch Expansion Joint with 80 mm max gap for LWR BG 52 kg on Concrete sleepers
RDSO/T-4165	Assembly for SEJ with 80 mm max gap for LWR BG 60 kg on concrete sleepers
RDSO/T-5748	Assembly for SEJ with LWR BG 60 kg (UIC) on PSC sleepers laid on curve with curvature from 0.5° to 1.5°
RDSO/T-6039	Assembly for SEJ with 190 mm max gap for bridge approaches for LWR BG 52 kg on concrete sleepers
RDSO/T-6263	Assembly for SEJ with 190mm max gap for bridge approaches for BG 60 kg(UIC) on PSC sleepers
RDSO/T-6257	Assembly for SEJ for 80 mm max gap with BG CR – 120 crane rails on PSC sleepers

(i) RDSO/T- 4160 and RDSO/T- 4165 are the conventional straight SEJs with 80 mm maximum gap. Each SEJ has a pair of tongue rails and stock rails, with 6 special sleepers to RDSO drawing No.RDSO/T-4149. All these are 300 mm wide sleepers with sleepers Nos 10 and 11



Typical SEJ layout

Fig. 3.4

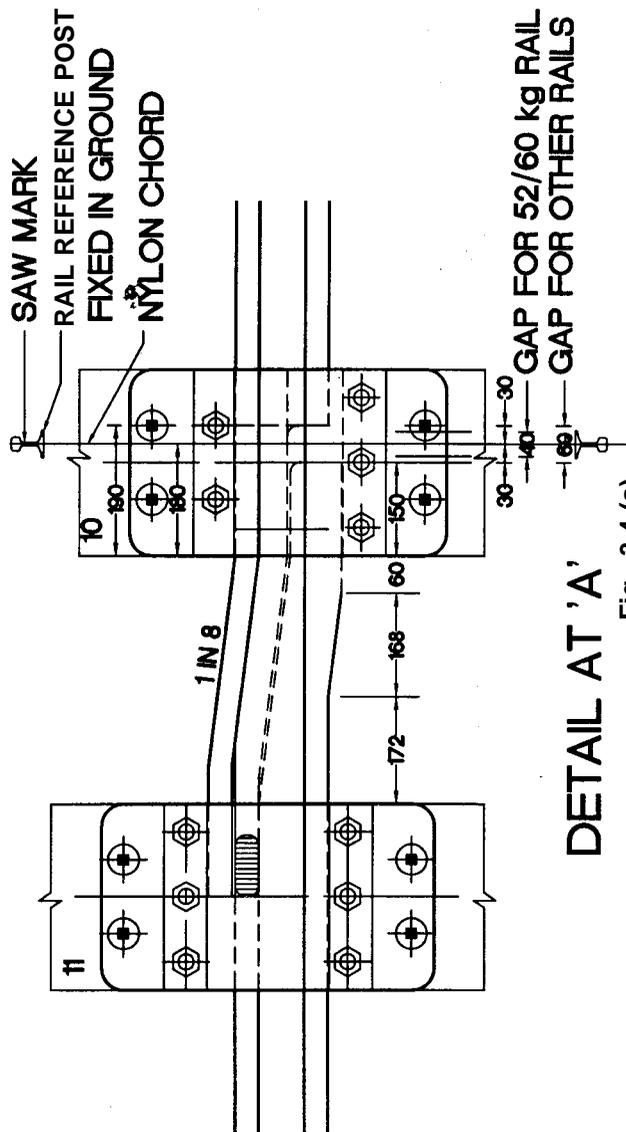


Fig. 3.4 (a)

(36)

with special fastenings and sleeper No. 8, 9, 12 and 13 with similar fittings. The centre line of sleeper No. 10 coincides with the tip of the tongue rail and the 40 mm initial gap is provided with the tip of the tongue rail coinciding with the centre of the sleeper No. 10. The centre to centre spacing of sleeper No. 10 and 11 is 700 mm while the sleepers spacing from 1 to 10 and 11 to 20 may be 600 mm or 650 mm depending upon the sleeper density. Fig. 3.4 gives details of a typical SEJ layout and 3.4(a) gives the details of location A.

(ii) RDSO/T-6039 & RDSO/T-6263 : These are wide gap SEJs for bridge approaches where the maximum gap permitted is 190 mm, the

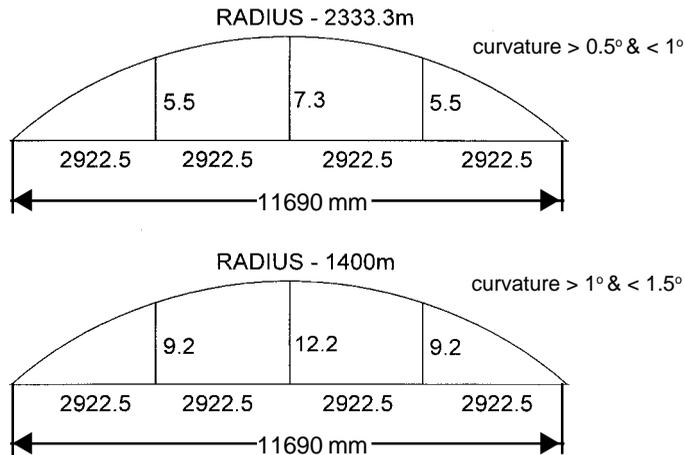


Fig 3.5

mean position is kept at 166 mm from centerline of sleeper No. 10 to enable the tongue rail to remain on the sleeper even when the entire expansion takes place. Sleepers No. 7, 8, 9, 10, 11, 12, 13 are special sleepers with sleeper nos. 9, 10 and 11 with special fastenings. Use is made of ERC Mark II clips with flat toe designed with a toe load of 350-400 kg/clip to enable free rail movement. Sleepers other than 7 to 13 are approach concrete sleepers with normal fittings. When gap is more than 100mm for passing diploories with smaller wheel diameter, use of an insertion piece in the gap should be made.

(37)

(iii) RDSO/T-5748 : These SEJ layouts can be used when the SEJ has to be laid in a curve sharper than 0.5° but not sharper than 1.5° . The tongue rail and stock rail are given curvature as given (Fig. 3.5).

The conventional SEJ design involves two bends in the stock rail and tongue rail which are locations of weakness resulting in fractures. Improved design SEJs developed by various industries are under trial on the Railways till final approval is received from the Railway Board. A brief description of these layouts is given below:

(1) **SEJ with one gap** : This design has been developed by M/s Rahee Industries Ltd, Calcutta. Fig 3.6 The design comprises of a pair of machined segments on the non-gauge face side of two non-bent running rails mounted with a gap between the juxtaposed rail ends and a third rail called a gap avoiding rail of predetermined length accommodated in the said machined segments parallel to and adjacent to the non bent straight length of the running rails. This rail is securely fitted to one of running rails with high tensile steel bolts. This running rail together with the gap avoiding rail is called the stock rail. The other running rail is called the tongue rail. The non-bolted segment of the gap avoiding rail braces the machined segment of the tongue rail.

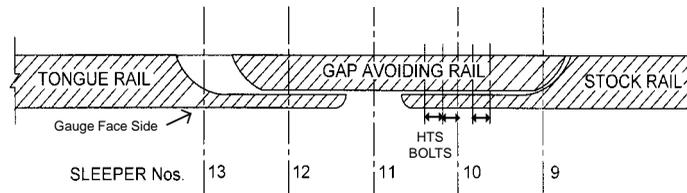


Fig 3.6 ONE GAP SEJ

Advantages :

- 1) No bends in tongue and stock rail.
- 2) Only 5 special sleepers of standard SEJ on PSC assembly are used.
- 3) Check rails guard against excessive play of worn out wheels.
- 4) Design suitable upto 180 mm maximum gap.

(2) SEJ with two gaps

Two designs have been developed by two different firms (M/s Bina Metalway, Jamshedpur and M/s Chintpurni Engineering Works, Barabanki). In both these designs two gaps of maximum 80 mm each are provided in one SEJ. Thus a maximum gap of 80 mm is available for an LWR on one side of the SEJ. Similarly a gap of 80mm is available for the LWR on the other side. The tongue rail is manufactured by cutting the rail at head and foot location. Two cut rails are joined together to make the stock rail.

Salient features of Bina Metalway 2-gap SEJ (Fig 3.7)

The stock rail is considered to be static with negligible expansion and contraction in length due to temperature changes. This SEJ makes use of 6 wider concrete sleepers each to Drg No. T/4149, with three sleepers located near each gap. The length of the SEJ is $5750 + 6950 + 5920 + 80 = 18700$ mm.

Hence a total gap of 18750 mm should be created while inserting this SEJ. The stock rail is fabricated out of two pieces of lengths 7140 mm and 5920 mm connected to each other by HTS bolts. While laying the SEJ it should be ensured that the ends of the stock rail are 40 mm away from the centre line of sleeper Nos. 12 and 22 with the tip of the tongue rail coinciding with the centre line of the sleeper.

1. Sleeper Nos. 1 to 31 should be at a spacing of 600 mm c/c.
2. Sleeper Nos. 10, 11, 12, 22, 23 and 24 are special sleepers to RDSO drawing No. T – 4149 and the rest are normal PSC line sleepers.

(39)

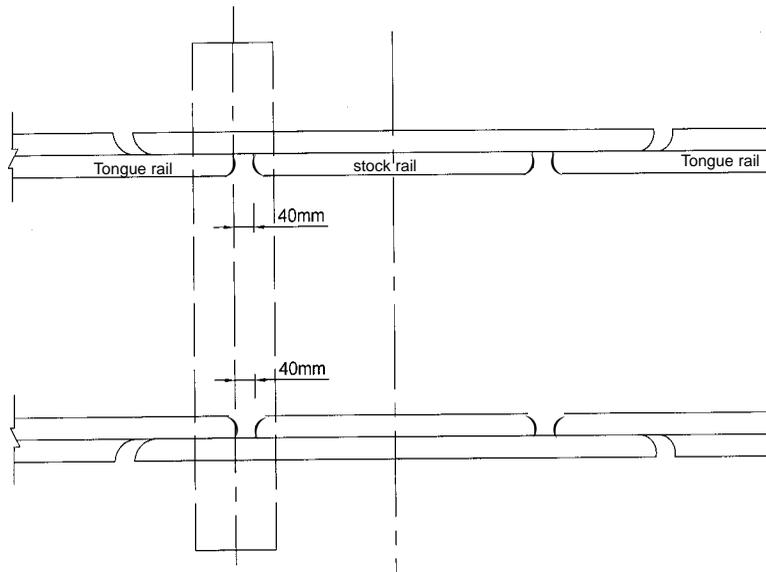


FIG 3.7 TWO GAP SEJ

3. Mean position of SEJ should be kept at centre line of sleepers No. 12 and 22.
4. The mean gap is 40 mm on each end.
5. The tongue rails are kept at mean position at centre line of sleeper Nos. 12 and 22, and stock rail end kept at 40 mm from mean position, thus creating a gap of 40 mm.
6. The mean position should also be marked on the rail posts erected on both sides of track.

3.3 Phenomenon of Hysteresis :

The behaviour of an LWR, as far as movement at the SEJ is concerned, is irrational which will be evident from the (Fig. 3.8) shown below :

As the temp. uniformly rises above 'O', the movement or expansion at the SEJ follows the movement – temp. rise curve OF where

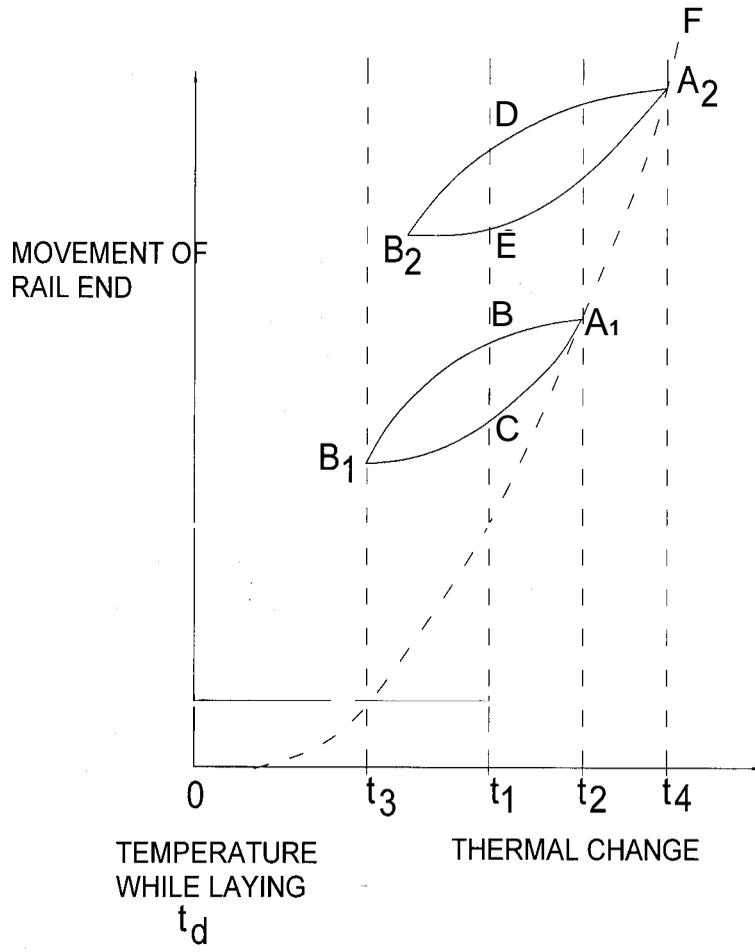


Fig 3.8 HYSTERESIS PHENOMENA

(41)

$$\Delta = \frac{AE(\alpha t)^2}{2R}$$

At any given temperature t_4 if the temp starts falling, then the movement at the SEJ does not follow the original path but traces out a new curve A_2DB_2 . If at B_2 the temp again reverses then the path traced out is B_2EA_2 rather than B_2DA_2 . Loops in the form $A_2DB_2EA_2$ are called hysteresis loops and are formed whenever there is a temperature reversal. In order to simplify matters, an annual hysteresis loop or curve is plotted which will envelope all the hysteresis loops formed on a daily basis. (Fig. 3.9)

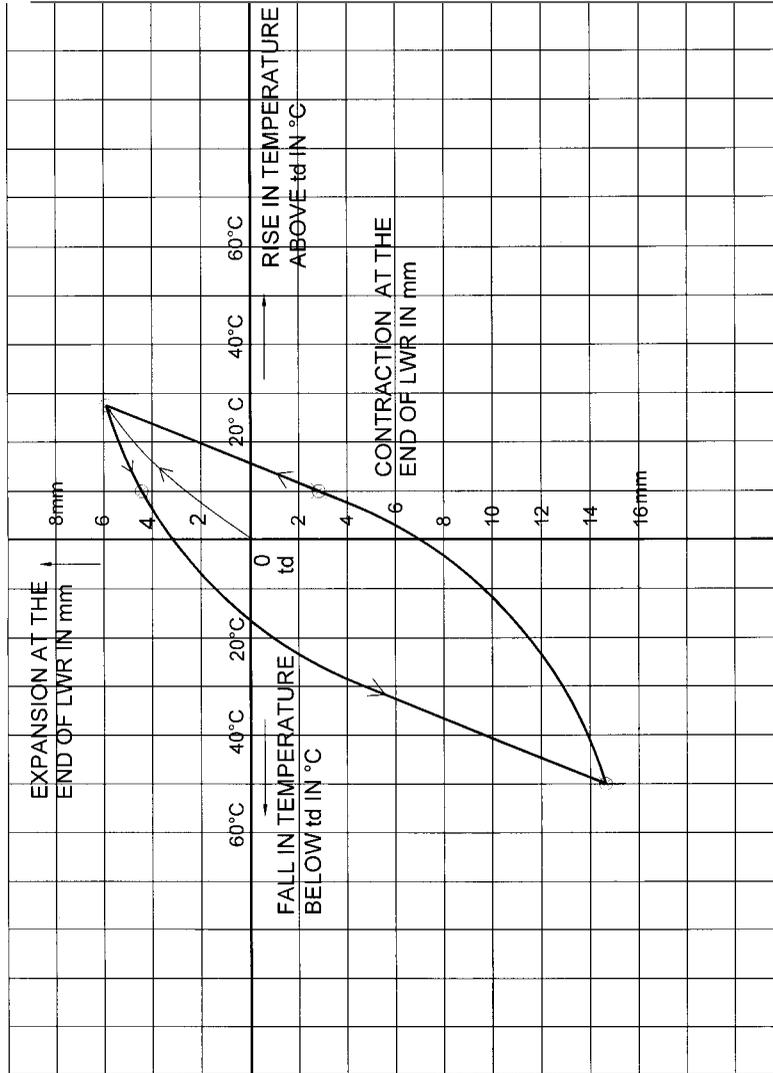
This hysteresis loop can be traced in two ways :

- 1) Temp rise from t_d to t_{\max} , fall from t_{\max} to t_{\min} and again a rise to t_{\max} from t_{\min} .
- 2) Temp fall from t_d to t_{\min} , rise from t_{\min} to t_{\max} and then a fall from t_{\max} to t_{\min} .

While plotting these curves it should be remembered that whenever there is a reversal of temperature, the longitudinal ballast resistance should be taken as twice its normal value ($2 R$ instead of R). The final hysteresis loop shall be an envelope of the two hysteresis loops as drawn above.

Implications of hysteresis:

At temp. t_p the movement at the SEJ may be an expansion equal to 'a' or a contraction 'b'. This would mean that the gap at the SEJ could be $(20 - a)$ or $(20 + b)$ if 20 mm is the initial gap. Hence, due to hysteresis the gap at SEJ is not a discrete value but a range. LWR Manual Annexure V gives the permissible range of gaps at the SEJ for different track structures at different rail temp. for different zones as given on the adjoining page :



ANNUAL HYSTERESIS LOOP
SHOWING THE MOVEMENTS OF ENDS OF L.W.R. PROVIDED
WITH SWITCH EXPANSION JOINTS

Fig. 3.9

(43)

SAMPLE TABLE FROM ANNEXURE V OF LWR MANUAL

GAPS BETWEEN REFERENCE MARK AND TONGUE RAIL TIP/
STOCK RAIL CORNER OF SEJ FOR VARIOUS TEMPERATURES
IN mm FOR BG, 52 Kg, PRC SLEEPER, 1540 Nos/km, VALUE OF
R (BALLAST RESISTANCE) ASSUMED = 13.28 Kg/cm/rail AND t_d
AS PER PARA 1.11

Temperature	Zone I	Zone II	Zone III	Zone IV
$t_d + 28$	–	–	–	14
$t_d + 25$	–	–	16	14 – 16
$t_d + 20$	–	17	16 to 18	15 – 19
$t_d + 15$	18	17 to 19	16 to 20	15 – 21
$t_d + 10$	18 to 20	18 to 21	16 to 22	16 – 23
$t_d + 5$	19 to 22	18 to 23	17 to 24	16 – 25
t_d	19 to 23	19 to 24	18 to 26	17 – 27
$t_d - 5$	20 to 24	19 to 26	19 to 27	18 – 28
$t_d - 10$	21 to 25	20 to 27	20 to 29	20 – 30
$t_d - 15$	22 to 26	22 to 28	21 to 30	21 – 31
$t_d - 20$	23 to 26	23 to 29	23 to 31	23 – 32
$t_d - 25$	24 to 27	24 to 29	24 to 32	24 – 33
$t_d - 30$	26 to 27	26 to 30	26 to 32	26 – 34
$t_d - 35$	27	28 to 30	28 to 33	29 – 34
$t_d - 40$	--	30	31 to 33	31 – 35
$t_d - 45$	--	--	33	33 – 35
$t_d - 48$	--	--	--	35

(44)

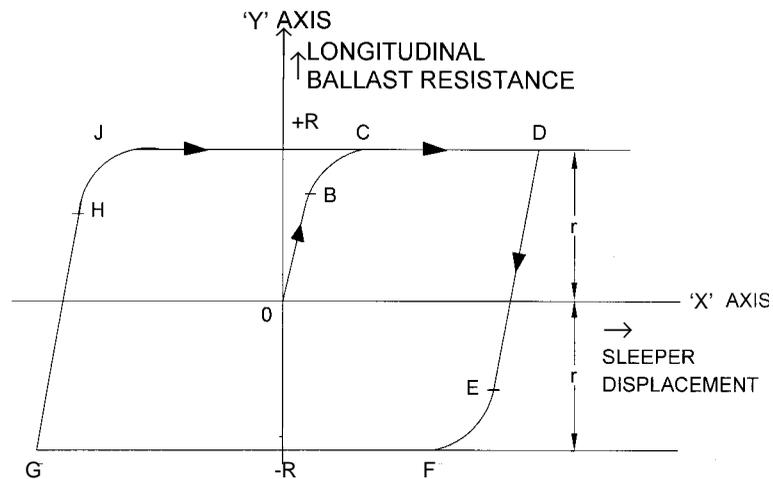


Fig. 3.10

The gaps are with an initial setting of 40 mm. When gaps have been initially laid at 60 mm, 10 mm should be added to each of these values. The gaps given in the above table are the theoretical gaps. A tolerance of $\pm 10\text{mm}$ is prescribed beyond the given gaps.

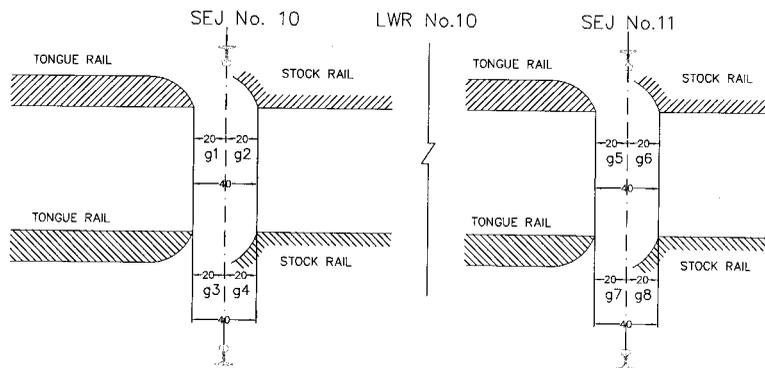
Cause of Hysteresis : Hysteresis is due to the behaviour of the longitudinal ballast resistance. A plot of the resistance offered by the ballast vis-à-vis the sleeper displacement is as given in Fig 3.10.

The ballast resistance first increases linearly as the sleeper displacement, then goes into the plastic zone and finally assumes a constant value R . If at this stage the temperature reverses then the value of the longitudinal ballast resistance drops to zero and then becomes $(-R)$ as shown above. This shows that at the time of reversal of temp the ballast resistance mobilized is $2R$. Due to this effect, the path traced out at the end of LWR follows an irregular path leading to the hysteresis phenomenon.

3.4 Gap measurements at an SEJ :

At the SEJ a reference line is established between the tongue rail and stock rail. This gap between the tongue rail and stock rail will be equal to 40mm for 52kg and 60kg rail sections, and for other rail sections 60mm as shown. (Fig. 3.11)

Gaps g1 and g2 are not discrete values but the permissible range has been defined in the LWR Manual Annexure V for different track structures, different zones and different prevailing temperatures. A sample page for filling up the movements observed at an SEJ as per annexure XIII A of the LWR Manual is shown in Fig. 3.12.



GAP MEASUREMENT AT AN SEJ

Fig. 3.11

Annexure XIII A
Chart of movement of LWR/CWR No.10

SEJs at the end of this LWR : SEJ NO.10 at km—; SEJ No.11 at km—

Date of Measurement	Time of Measurement	Rail Temp	Right or Left Rail	Distance (mm) between tongue/stock rail & Reference line at SEJ No.10		Distance (mm) between tongue/stock rail & Reference line at SEJ No.11		c/c space between sleepers the two central sleepers	Measured by	Rectification carried out	Remarks
				Observed (a)	Permissible Range	Observed	Permissible Range				
6.10.2004	10:00	30°C	Left rail Right rail	g2 g4				at SEJ No. 10			
6.10.2004	14:00	40°C	Left rail Right rail		g5 g7			at SEJ No. 11			

Fig 3.12- Sample of SEJ gap measurements. Also refer Fig 3.11

CHAPTER IV

PERMITTED LOCATIONS AND TRACK STRUCTURE

4.1. General Considerations :

1. It is a policy that all new constructions, gauge conversions, doublings and permanent diversions will be opened with LWR wherever permissible by the provisions of the LWR manual.
2. All complete track renewals (primary) shall provide for LWR/CWR wherever permissible. Existing rails on permitted locations shall be converted into LWR/CWR provided they meet the requirements laid down in the Manual for Welding of rail joints by Alumino thermit (SKV) process.
3. In goods running lines, goods yards, reception yards marshalling and classification yards, rail joints may be welded to form LWR if the condition of all components of the track is generally sound and without any deficiency subject to such relaxation as may be approved by Chief Engineer, in each specific case.

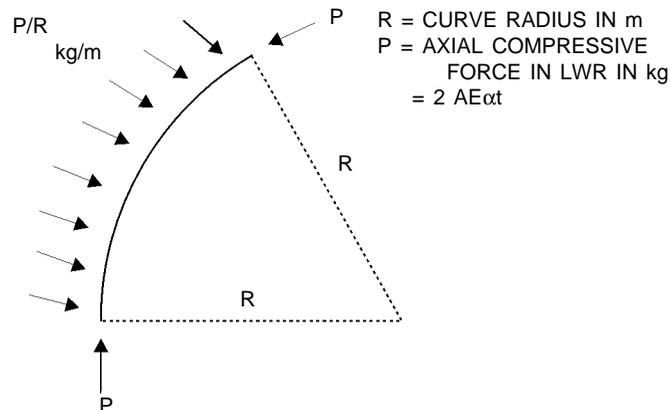


Fig. 4.1 External Equilibrium of Curved LWR Track

4.2. Alignment

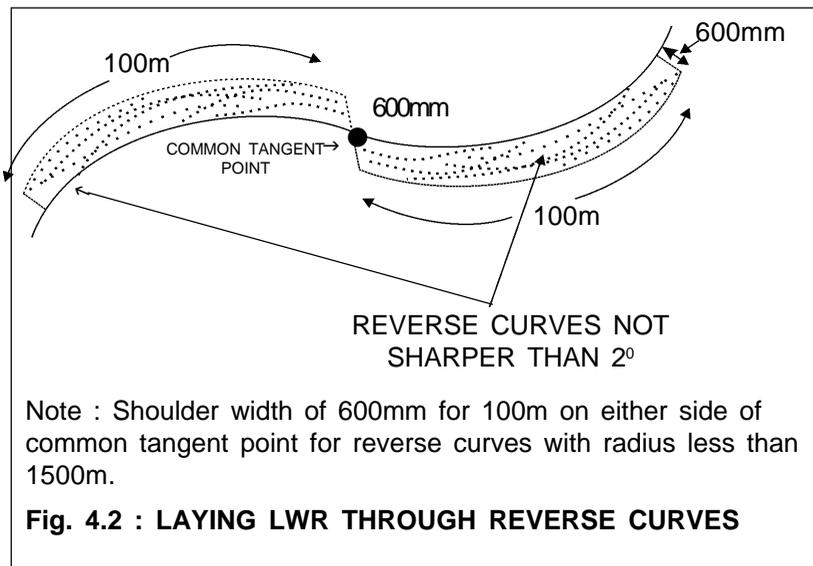
1. LWR/CWR shall not be laid on curves sharper than 440 meters radius both for BG and MG.

As indicated in Fig. 4.1 the external equilibrium of a curved elastic beam of radius R subjected to a longitudinal force 'P' requires a continuously distributed external force of magnitude 'f'

where $f = \frac{P}{R} \text{ kg/m}$. This will be derived from the lateral ballast

resistance τ and $\tau - \frac{P}{R}$ is the effective lateral resistance against

buckling danger. In order to ensure that the stability of the LWR in curve is the same as in straight the lateral ballast resistance in curve should be made high by at least P/R kg/m. Hence a larger



shoulder width on curves and a restriction on the degree of curvature is prescribed. On Broad Gauge a shoulder ballast width of 500mm has been adopted.

(49)

2. LWR/CWR may be continued through reverse curves not sharper than 875 metres. For reverse curves sharper than 1500 meters radius, shoulder ballast of 600 mm over a length of 100 m on either side of the common point should be provided. These details are shown in Fig. 4.2.

4.3. Gradients :

1. The steepest grade permitted is 1 in 100 for LWR sections. This is because steeper grades imply larger longitudinal forces due to traction and braking which would be detrimental to the health of the LWR causing an increase in the longitudinal stresses in the rail.
2. A vertical curve shall be provided at the junction of grades when the algebraic difference between the grades is equal to or more than 4mm per metre or 0.4% as laid down in para 419 of IRPWM. This vertical curve will serve to smoothen the geometrical transition and introduce a gradual change in the direction of longitudinal force as shown in Fig.4.3.
3. Vertical curves should be of adequate radius as indicated in the table below

Broad Gauge		Metre Gauge	
Route	Minimum Radius	Route	Minimum Radius
A	4000 m	All routes	2500 m
B	3000 m		
C, D & E	2500 m		

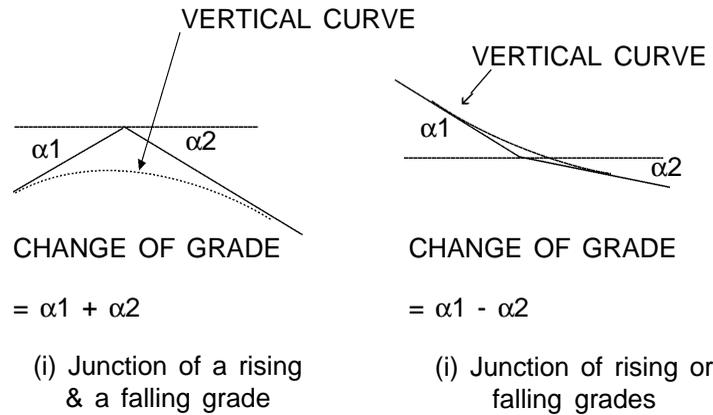


Fig. 4.3 Provision of Vertical curves at grade Intersection Points

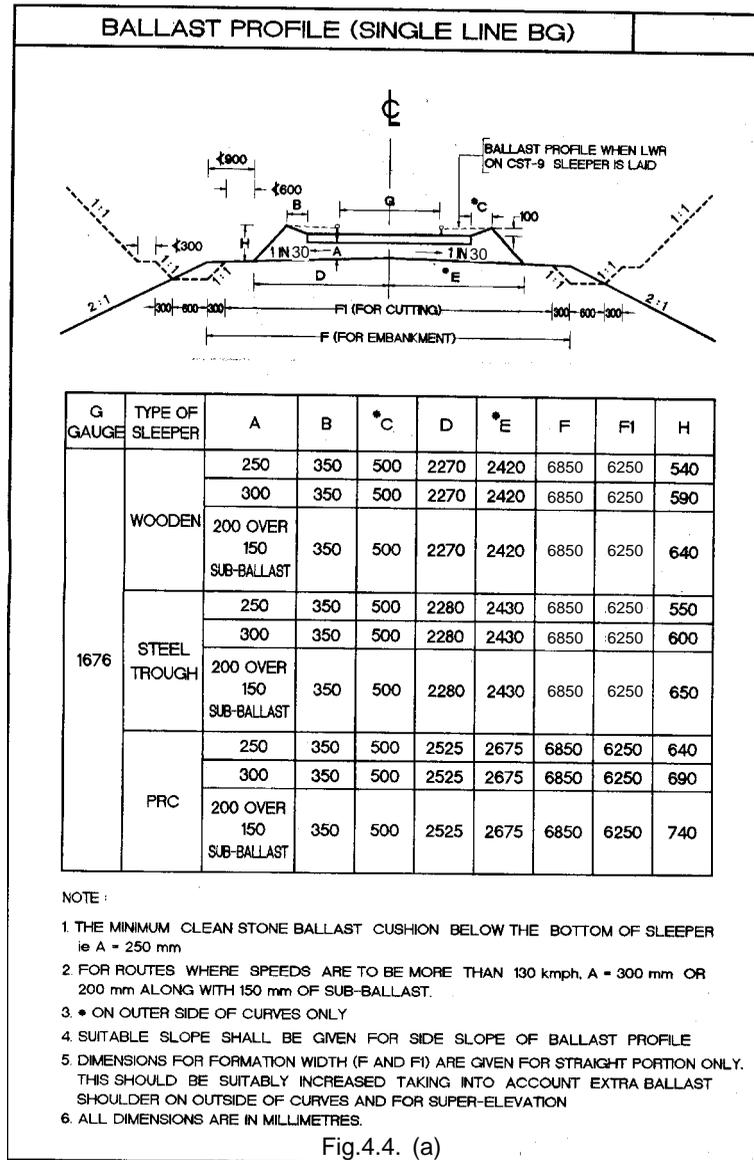
4.4 TRACK STRUCTURE

4.4.1. Formation :

LWR/CWR should be laid on stable formation. Yielding formations/troublesome formations should normally be isolated from the LWR section by laying SEJs on either side. Yielding formations will result in differential settlement and deformation of track causing misalignment and unevenness. The stability of the LWR will be affected and buckling of the LWR may result. The formation width is critical to avoid rolling down of ballast in banks.

4.4.2. Ballast Cushion and Section :

At least 250mm clean ballast stone cushion should be available below the sleeper. For speeds above 130kmph on BG and 100 kmph on MG, a 300mm clean ballast cushion or a 200mm clean ballast cushion with 150 mm of sub-ballast has been prescribed. The ballast section for the LWR is a heaped up section with heaping up of 100mm starting from the edge of the sleeper. The shoulder ballast width is increased to 350mm from the standard shoulder width of 300mm. In case of curves the shoulder ballast width prescribed is 500 mm. Details of ballast profile in bank and cutting for BG & MG in single and double lines are given in Fig.4.4.(a),(b),(c) & (d).



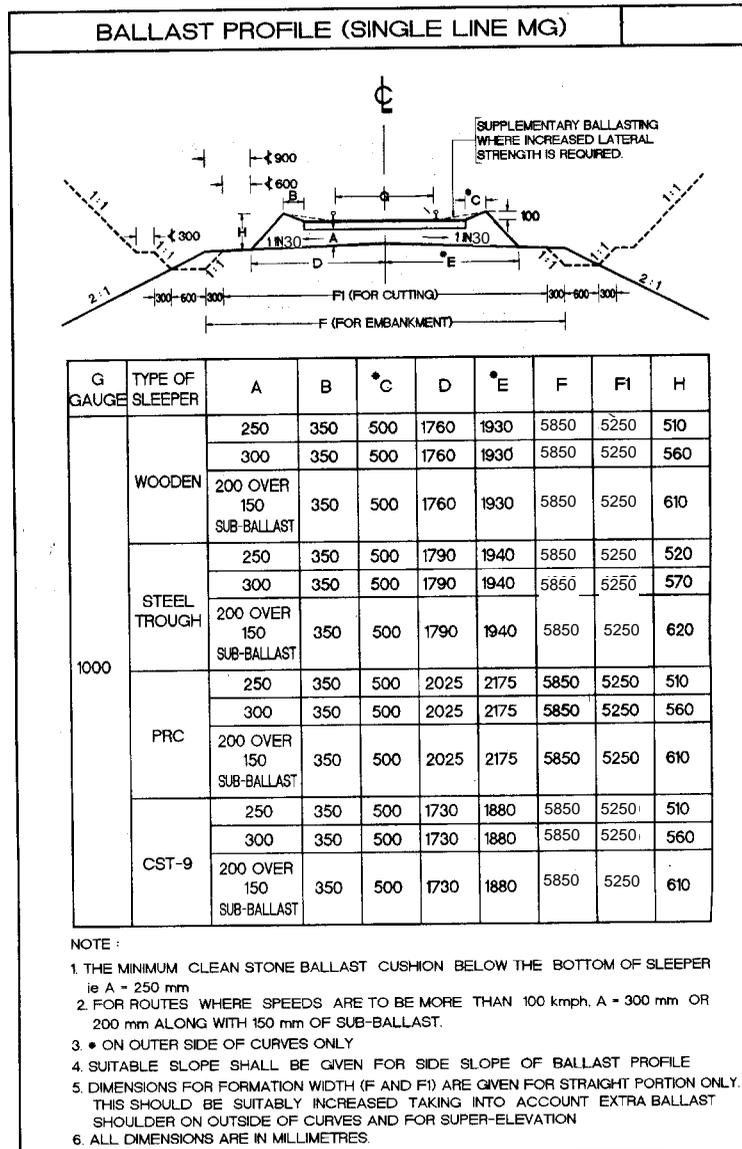


Fig.4.4. (b)

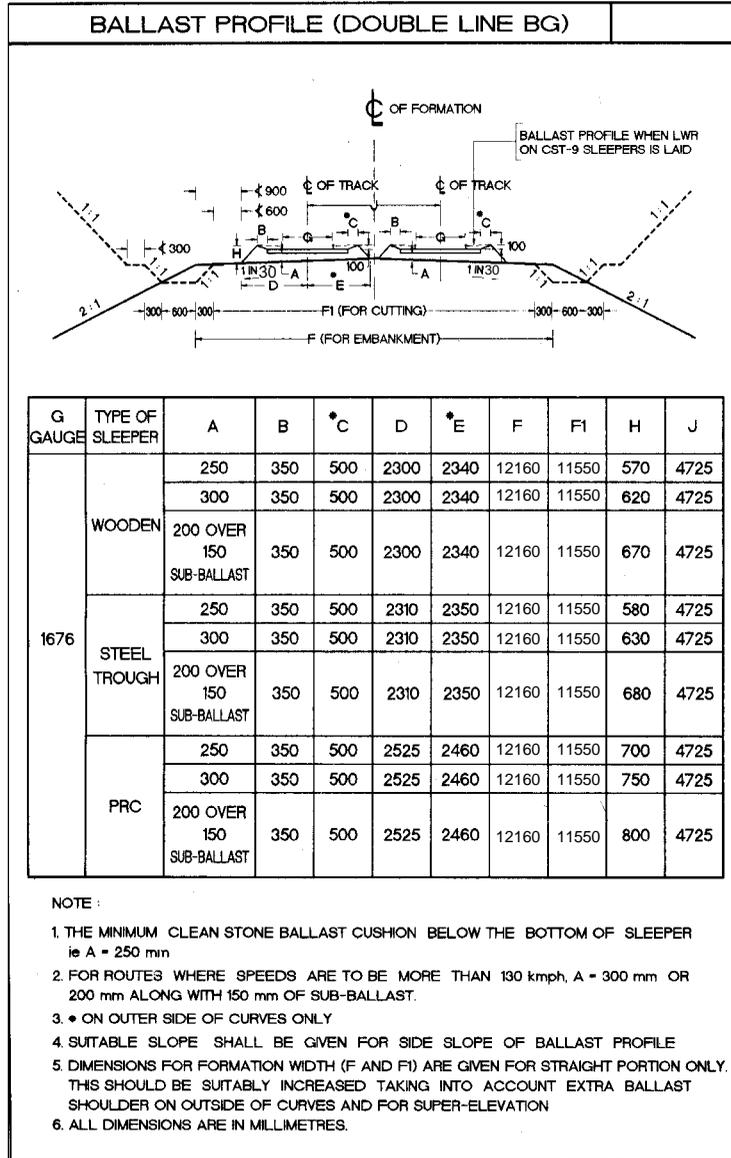


Fig.4.4. (c)

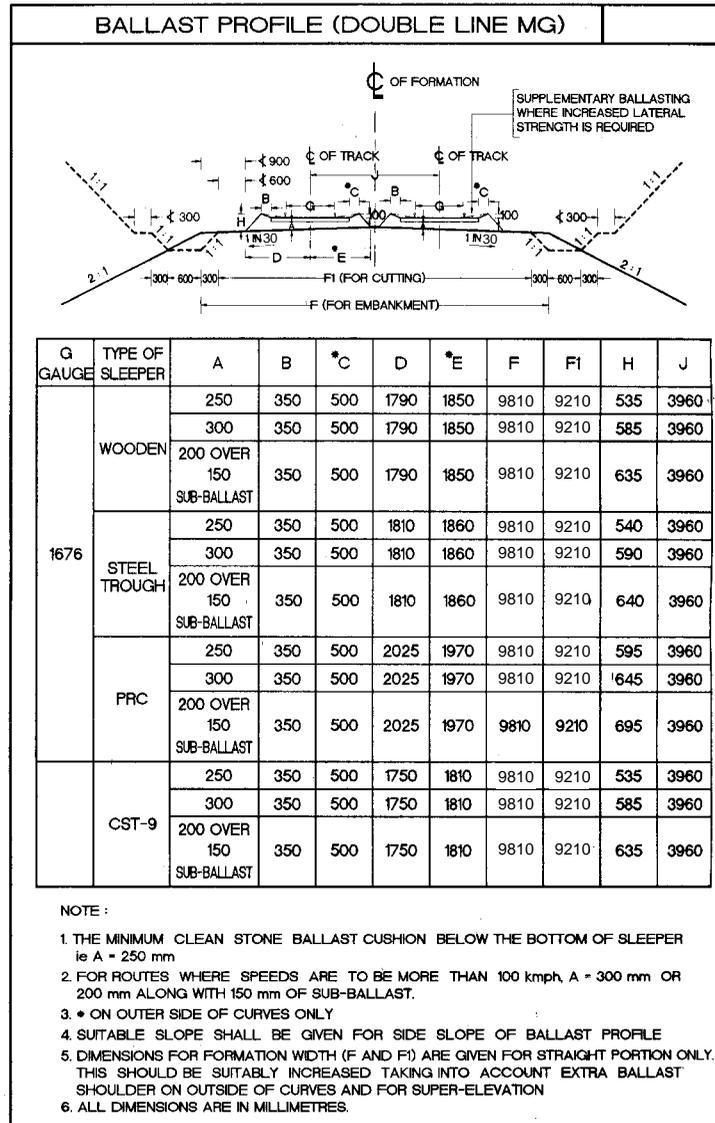


Fig.4.4. (d)

4.4.3 Sleeper and fastenings :

As explained earlier, the entire theory of the LWR is built on the premise that there is no relative slip of the rail with respect to the sleeper, and any movement which occurs is with rails and sleepers moving together. To achieve this the following types of sleepers and fastenings have been approved for use in LWR/CWR :-

BG

- (i) Concrete sleepers with elastic fastenings.
- (ii) Steel trough sleepers with elastic fastenings for speeds not exceeding 130 kmph (elastic fastenings are used on steel trough sleepers with modified loose jaws or with a steel pad plate welded to the steel sleeper.) As an interim measure speeds upto 160 kmph have been permitted with such arrangements.

Some exceptions to the above recommendations are as below :

- (i) Steel sleepers with 2 way keys and CST-9 sleepers with 2 way keys are permitted for speeds upto 130 kmph, provided no maintenance problem is faced and performance is satisfactory.
- (ii) On steel trough sleepers with key fastenings, the breathing lengths shall preferably be provided with elastic fastenings.
- (iii) Special precautions to be taken for CST-9 track are as under :
 1. On single line section keys on adjacent sleepers to be driven in opposite directions :
 2. In double line sections 75% of the keys to be driven in the direction of traffic and 25% in the opposite direction. In the breathing length, however, adjacent sleepers will have keys driven in the opposite directions.

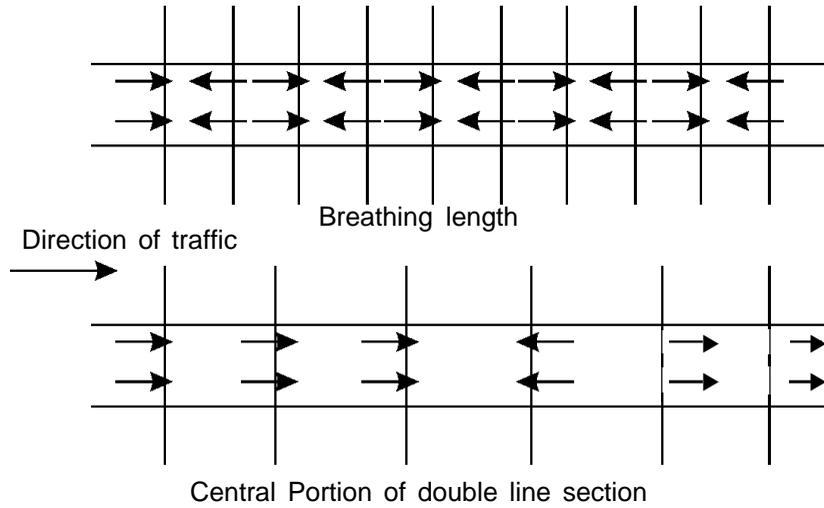


Fig 4.5 Direction of key driving with CST-9 sleepers.

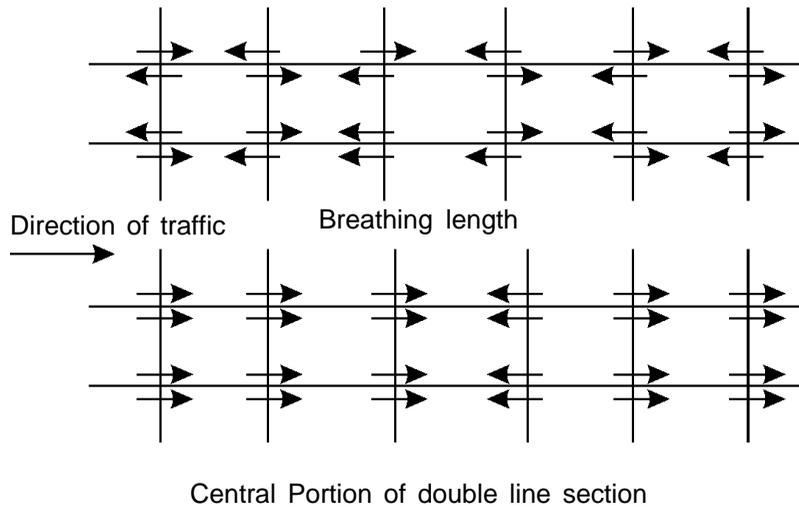


Fig 4.6 Direction of key driving with steel sleepers.

(57)

The direction of key driving for CST-9 and steel trough sleepers in LWR territory for single and double line sections is given in Fig 4.5 and 4.6 respectively.

MG : The recommended sleepers for speeds above 75kmph but a must for speeds above 100 kmph are :

1. Concrete sleeper with elastic fastenings.
2. Steel trough sleepers with elastic fastenings.

The recommended sleepers for speeds upto 100kmph are:

1. Steel sleepers with two way keys.
2. CST-9 sleepers with keys.

Wooden sleepers with anti-creep bearing plates and two way keys or elastic fastening may be permitted, for continuing LWR, if behaving satisfactorily for a maximum speed of 130 kmph in BG and 100 kmph in MG.

Sleeper density :

The minimum sleeper density (number of sleepers/km) in LWR/ CWR shall be as follows :-

Type of sleeper	Sleeper density in BG/MG
i) PRC	1310 in temperature zone I & II
ii) PRC	1540 in temperature zone III & IV
iii) Other sleepers	1540 in all temperature zones

4.4.4 Rails : The following rail sections can be welded to form an LWR :

BG	MG
90R	75R
52kg	90R
60kg	

(58)

1. In MG, 60 R rails converted into LWR may be permitted to continue if showing satisfactory performance.
2. In the same LWR, different rail sections are not permitted. This is because of the following reasons:
 - (i) Thermal forces generated in rails of different cross sectional areas are different. This makes the behavior of the LWR non uniform. The destressing temperatures are also different for 52kg and 90R rails.
 - (ii) While permitting two different rail sections in an LWR, combination welded joints cannot be avoided. As the gauge faces have to be matched, eccentricity is induced in the axial forces, resulting in additional stresses in the rail.
 - (iii) The ultrasonic flaw detection of combination welds is not completely foolproof.

The track structure suggested at the junction of a 52kg and 60kg LWRs is shown in Fig.4.7. The suggested track structure meets with the requirement of continuing the same track structure for three rail lengths beyond the SEJ.

3. Before converting an existing fish plated track into LWR/ CWR, following precautions should be taken :

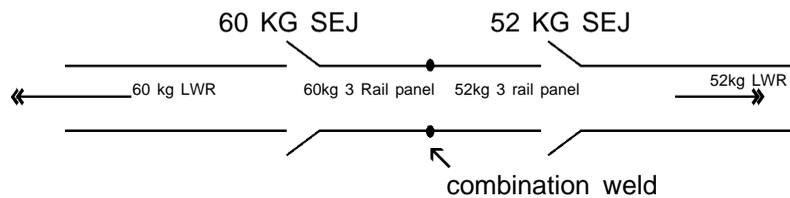


Fig.4.7

(59)

- i) Rails to be ultrasonically tested and all defective rails replaced.
 - ii) Rail ends which are bent, hogged, battered or having a history of bolt hole cracks should be cropped before welding. Cropping by 300mm – 450mm is generally done.
 - iii) Rails should have a residual life of more than 10 years.
4. New rails used in LWR/CWR shall be as far as possible without fish bolt holes. If the rail ends have to be joined during installation then use should be made of 1 m long fishplates/ normal fishplates with screw clamps with SRs of 30 kmph with 1 m long fishplates and 20 kmph with normal fishplates.

4.4.5 MISCELLANEOUS

1. Level X-ings should not fall within the breathing lengths of the LWR as level crossings are rigid structures and will not permit thermal movements of the breathing length to take place.
2. Normally LWRs are not taken through points and crossings. For concrete sleeper track a 3-rail panel should be provided between the SEJ and stock rail joint. Similarly a 3-rail panel is required to be provided between the heel of crossing and SEJ. The point and crossing assembly has a distinct track structure with different maintenance schedules and subject to large lateral forces when a train negotiates a turnout. Due to this reason, it was considered desirable to isolate the point and crossing assembly from the LWR. However with the introduction of stress frames in the point and crossing area or taking other special measures it is now possible to carry an LWR through station yards including points and crossings.
3. Insulated Joints for track circuiting in LWR/CWR shall be provided using glued joints of the G3(L) type. The G3 (L) type glued joint has a pullout capacity of 150 tons for 52 kg rails and 175 tons for 60 kg rails and an insulation

resistance not less than 25 megaohms in dry condition or 3 kiloohms under saturated conditions. Glued joints are considered as track components whose procurement has to be done by the engineering department.

4. Location of SEJs : The exact location of the SEJ will be fixed taking into account location of various obligatory points such as level crossings, girder bridges, points and crossings, gradients and curves. SEJ with straight tongue and stock rail shall not be located on curves sharper than 0.5 degree (3500m radius). SEJ shall also not be located on transition portion of cuves.

4.5. LWR on Bridges :

- (i) Bridges with ballasted decks (without bearing) : LWR/CWR can be continued over bridges without bearings like slabs, box culverts and arches.
- (ii) Bridges with/without ballasted deck with bearings : When the bridge structure and the track exhibit movement relative to each other then there are interaction effects which have to be taken into consideration. These interaction effects have been discussed in UIC774-3R (Reference 9) some parts of which have been given below.

4.5.1 LWR on bridges: Track Bridge Interaction(UIC 774-3R)

1. Introducing a bridge under a CWR track means effectively that the CWR track is resting on a surface subject to deformation and movements hence causing displacement of the track. Given that both track and bridge are connected to one another either directly or through the medium of ballast and are able to move, any force or displacement that acts on one of them will induce force in the other.
2. All actions which lead to interaction effects are those that cause relative displacement between the track and the deck. These are :-
 - i) The thermal expansion of the deck only in the

(61)

case of the CWR or the thermal expansion of the deck and of the rail whenever a rail expansion device is present.

- ii) Horizontal braking and acceleration forces.
- iii) Rotation of the deck on its supports as a result of the deck bending under vertical traffic loads.
- iv) Deformation of the concrete structure due to creep and shrinkage.
- v) Effects of temperature gradient.

Out of these 5 factors, the first 3 are more important.

3. The forces created due to interaction between track and bridge are dependent on a number of parameters of bridge and track both :

The bridge parameters affecting the interaction forces are :

(1) Expansion length of the bridge(L): For a single span simply supported bridge the expansion length is the span length. For a continuous bridge with a fixed support at the end, it is the total length of the deck. If the fixed elastic support is located at some intermediate point, the deck is considered to have two expansion lengths on either side of fixed elastic support.

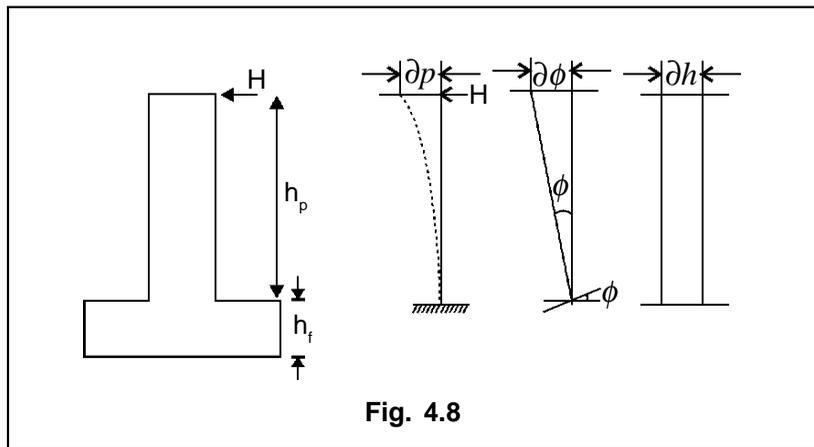
(2) Support stiffness : The resistance of the deck to horizontal displacement is a fundamental parameter as it affects all interaction phenomena. This factor is determined primarily by the total stiffness of the supports. The total support stiffness is composed of the stiffness of each support. The stiffness of each support is in turn composed of the stiffness of the bearing, pier, base, foundation and soil. The stiffness K of the support including its foundation to displacement along the longitudinal axis of the bridge is given by

(62)

$$K = \frac{H(KN)}{\sum \partial i(cm)}$$

with $\partial i = \partial p + \partial \phi + \partial h + \partial a$

where, ∂p = displacement at the head of the support due to



deck's deformation (this could be calculated assuming the pier to be a cantilever fixed at the base)

$\partial \phi$ = displacement at the head of the support due to foundation rotation.

∂h = displacement due to horizontal movement of the foundation.

∂a = relative displacement between upper and lower parts of the bearing

The value of the displacement component is determined at the level of the bearing as shown in Fig 4.8.

3) Bending stiffness of the Deck : As a result of bending of the deck the upper edge of the deck is displaced in the horizontal direction. This deformation also generates interaction forces.

4) Height of the Deck : The distance of the upper surface of the deck slab from the neutral axis of the deck and the distance of neutral axis from the centre of rotation of piers affects the interaction phenomena due to bending of the deck.

TRACK PARAMETERS : The resistance 'k' of the track per unit length to longitudinal displacement 'u' is an important parameter. This parameter in turn depends on a large number of factors such as whether the track is loaded or unloaded, ballasted or frozen, standard of maintenance etc. The resistance to longitudinal displacement is higher on loaded track than on unloaded track as can be seen from Fig. 4.9. The value of k has to be established by each railway system as per its track structure.

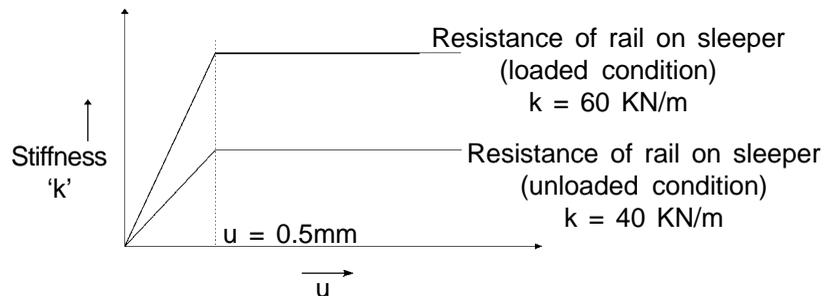
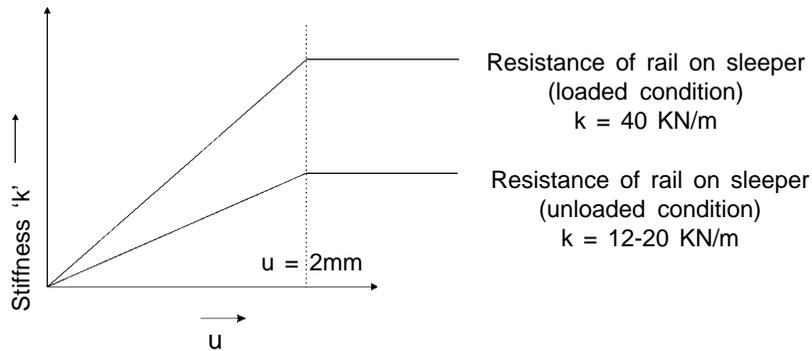


Fig. 4.9 TRACK STIFFNESS PARAMETERS (FROZEN BALLAST)

Once the values of K, the stiffness of the bridge structure and k, the stiffness of the track have been evaluated, use can made of the interaction diagrams given in UIC774-3R for calculation of the additional stresses in the rail and additional forces at the bridge support due to each of the actions causing interaction effects: namely (1) change of temperature (2) acceleration and braking forces (3) deck deformation.

(64)



**Fig. 4.9 TRACK STIFFNESS PARAMETERS
(NORMAL BALLAST)**

1 Changes in temperature :

It is assumed that there is change of temperature of $\pm 35^{\circ}\text{C}$ from the reference temperature for the bridge while for the rail could deviate by $\pm 50^{\circ}\text{C}$. Due to change of temperature additional stress will develop in the rail and additional force at the support. These are obtained from the interaction charts given in UIC774-3R.

2. Actions due to braking and acceleration

The braking and acceleration forces applied at the top of the rail are assumed to be distributed over the length under consideration with the following standard values :

Acceleration = 33 KN/m per track

Braking = 20 KN/m per track

These values could be modified to take into account the longitudinal loadings given in the Bridge Rules.

3. Actions due to bending of deck

Vertical traffic loads cause the deck to bend, which in turn causes rotation of the end sections and displacements of the upper edge of the deck. The design curves for the evaluation of the interaction due to vertical bending of the bridge deck have been evaluated

(65)

with respect to the standard longitudinal plastic shear resistance equal to 20 KN/m and 60 KN/m for unloaded and loaded track respectively.

The design curves are given for the following two different situations :

- deck bridge – the track lies on the top of the bridge deck (deck neutral axis below track axis)

- through girder bridge – deck neutral axis above track axis.

Combining load cases

For calculation of the total support reaction and in order to compare the global stress in the rail with the permissible value set by each railway, the global effect

$\sum R$ is calculated as follows :

$$\sum R = \alpha R(\Delta T) + \beta R(\text{braking}) + \gamma R(\text{bending})$$

α, β, γ are the combination factors.

Permissible additional stresses in continuous welded rail on the bridge

Theoretical stability calculations on UIC 60kg CWR of a steel grade giving at least 900 N/mm² strength, minimum curve radius 1500 m, laid on ballasted track, with concrete sleepers and consolidated ballast cushion greater than 30 cm give a total possible value for the increase of rail stresses due to track/bridge interaction as indicated below:

The maximum permissible additional compressive rail stress is 72 N/mm²,

The maximum permissible additional tensile rail stress is 92 N/mm².

For structures consisting of one deck, the values of the interaction effects can be calculated by using the design graphs in Appendix 'A' – page 36 and Appendix 'B' page 42 give in UIC report 774 – 3R.

4.5.2. Provisions given in the LWR Manual for carrying LWR over bridges:

However, for a simple understanding of the problem let us consider the effect of thermal variation alone as the cause of interaction between the girder and the LWR. As a result of thermal variation the girder has a tendency to expand or contract being provided with bearings. On the other hand the central portion of the LWR is fixed in position irrespective of the temperature changes that occur. This results in an interplay of forces between the girder and the LWR, the magnitude of the force being dependent upon the nature of fastenings being provided between the rail and sleeper. To clarify this aspect of interplay of forces between rail and girder, consider the case of a girder bridge provided with fastenings between the rail and sleeper with a creep resistance equal to 'p' kg per rail seat. The bridge sleepers are rigidly fixed to the top flange of the girder by means of hook bolts. On variation of temperature due to the creep resistance of the fastenings, free expansion/contraction of the girder is prevented. Consequently additional forces are developed both in the girder as well as in the rail. The magnitude of this force developed depends upon the value of 'p' (the creep resistance) and orientation/nature of the bearings provided in each span of the bridge.

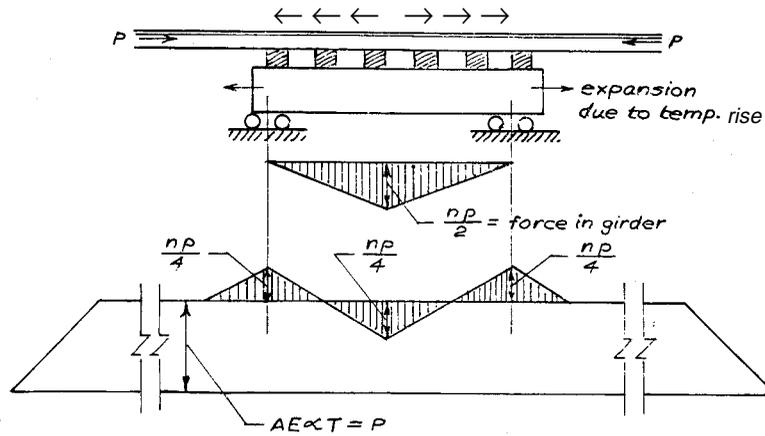
The following cases have been considered:

- Single span bridge :
1. One end fixed, other end free.
 2. Both ends of girder with free bearings.
- Multiple span bridge:
1. One end fixed and the other free with dissimilar bearings on a pier
 2. One end fixed and the other free with similar bearings on a pier
 3. free bearings at both ends.

The forces developed in the rail and girder for each of the five cases mentioned above are given in Fig. 4.10. These LWR force diagrams indicate that :

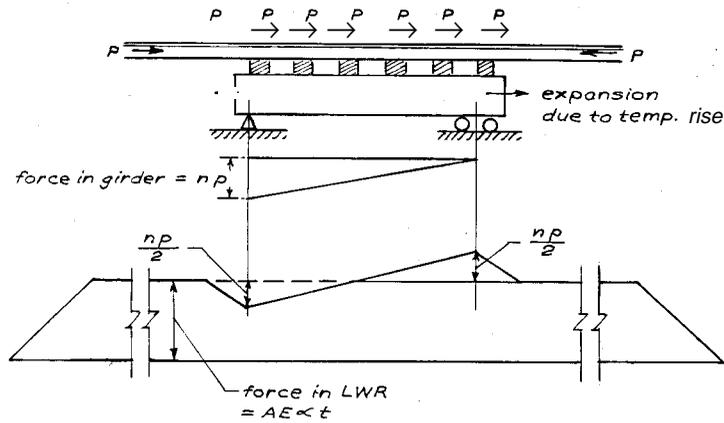
i) For sliding bearings at both ends of the girder, the increment of

force in the LWR is $\frac{np}{4}$, where 'n' is the number of sleepers per span with creep resistant fastenings and 'p' is the creep resistance



n = No. of sleepers per span
 p = creep resistance per rail seat

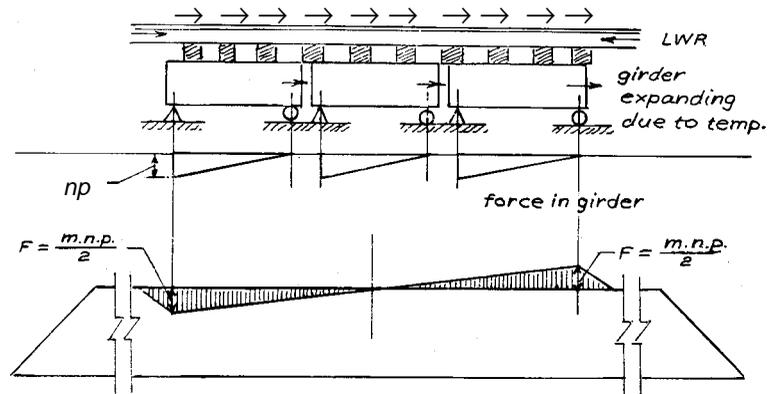
Fig. 4.10 (a)



n = No. of sleepers per span
 p = creep resistance per rail seat

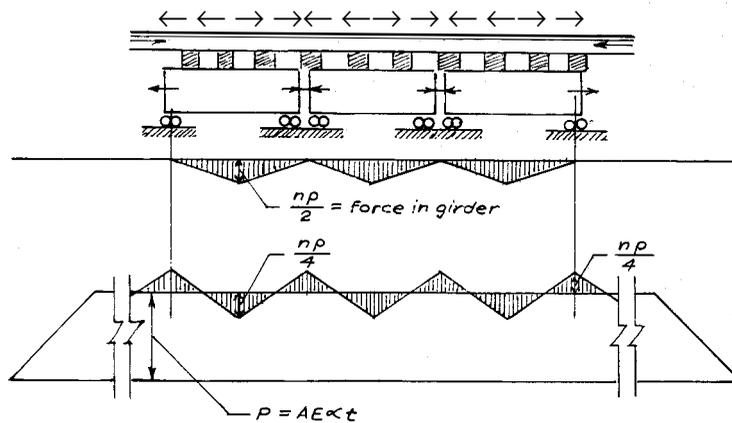
Fig. 4.10 (b)

(68)



m = No. of spans
 n = No. of sleepers per span
 p = creep resistance per rail seat

Fig. 4.10 (c)



n = No. of sleepers per span
 p = creep resistance per rail seat

Fig. 4.10 (d)

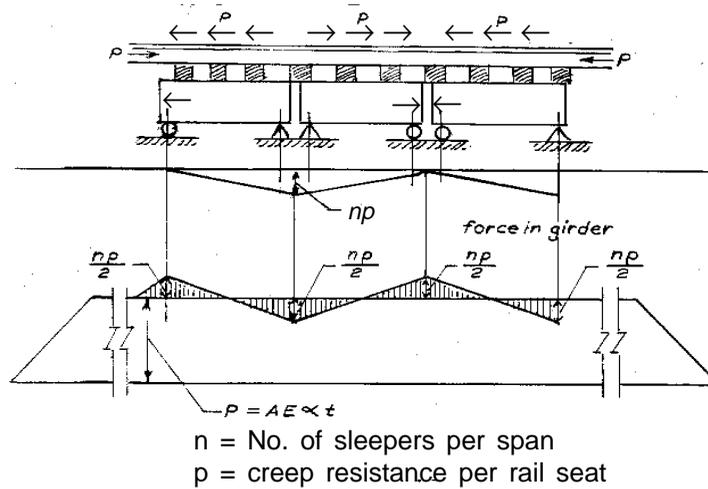


Fig. 4.10 (e)

per rail seat (4.10(a)). This increment of force will remain the same irrespective of the number of spans of the bridge (4.10(d)).

ii) In girders with one end fixed and the other end free the

increment of force in the LWR at the roller end is $\frac{np}{2}$ for a single

span bridge, where n = number of sleepers in the span with creep resistance of 'p' kg per rail seat (4.10(b)). If it is a multiple span bridge with 'm' number of spans, the increment of force in the

LWR at the roller end will be $\frac{m \times n \times p}{2}$. The resultant LWR force

diagram is shown in the sketch (4.10(c)). This is the case when on a pier bearing for one girder is a fixed bearing while the bearing of the other girder is a free bearing.

iii) There could be a situation where a pier supports similar nature bearings i.e. the bearings of the two girders are either fixed or free. In this case there will be no cumulative build up of force and the resultant LWR force diagram will be as indicated in Fig.4.10(e).

(70)

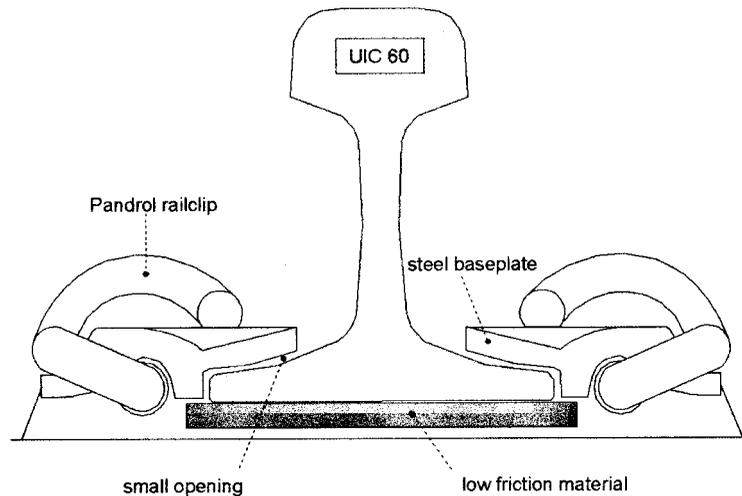


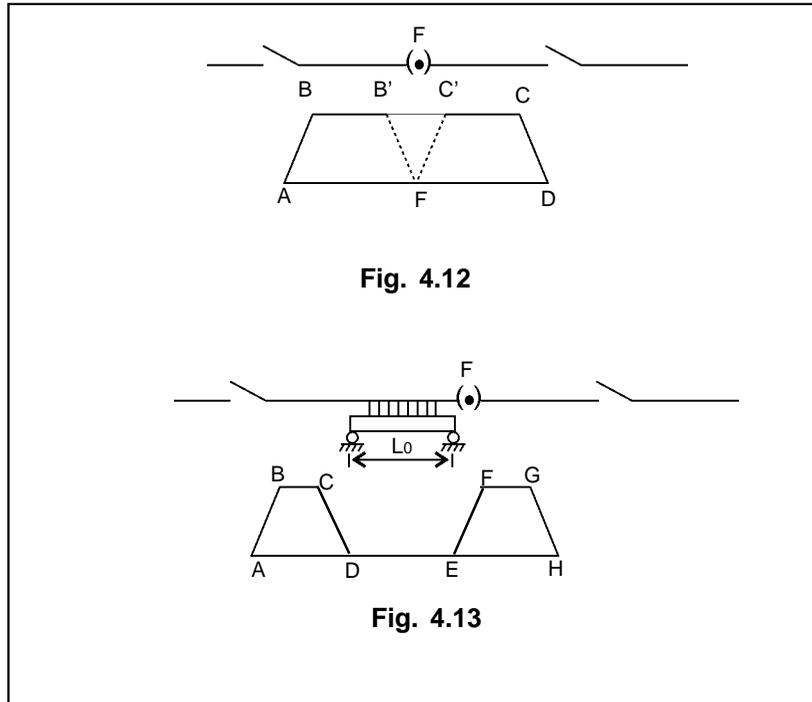
Fig. 4.11

In order to avoid interplay of forces between the LWR and girder a possible solution would be to provide rail free fastenings between rail and sleeper on the girder bridge. It is with this assumption that the provisions for laying an LWR over bridges have been framed in the LWR manual.

Fastenings used to connect the rail to the sleeper could be of two types : (1) Creep resistant fastenings and (2) Rail free fastenings which are now termed as zero longitudinal restraint fastenings. RDSO Report No. C-169 investigates the creep resistance offered by different types of rail –sleeper fastenings.

On the Indian Railways we have been traditionally using dog spikes and rail screws as rail free fastenings although now Pandrol has come up with a zero longitudinal restraint design(Fig 4.11). Under normal circumstances there is a small gap between the base plate (steel) and the top side of the rail foot. In case of large lateral forces, the baseplate prevents the overturning of the rail. The pad under the rail is made up of low friction material like teflon, which provides an almost zero friction

(71)



movement between the rail and sleeper.

Use of rail free fastenings on bridges where LWR is proposed to be used is now mandatory due to requirement of minimizing the interaction of forces between the LWR and the girder. However, this results in another problem : enhanced gap at fracture, when the fracture occurs on the approach of bridge laid with LWR.

Consider an LWR laid on normal formation with the usual force diagram A B C D. in the event of fracture at location 'F' the stress in the LWR is released at that location and two new breathing lengths B¹F and C¹F are formed on either side of the fracture location. (Fig 4.12)

(72)

The gap g_1 at the fracture location will be given by

$$g_1 = \frac{AE(\alpha t)^2}{2R^l} \times 2 \text{ ————— (1)}$$

[Assuming equal movement on either side of F]

R^l represents the longitudinal ballast resistance mobilised at the time of the fracture, which is generally about 50% to 60% of the normal R value, due to the sudden nature of occurrence of a fracture.

However, if the same fracture had occurred in the approach of a bridge provided with LWR and rail free fastenings the modification of the force diagram will be as given in the figure 4.13.

In this figure, ABCDEFGH represents the altered force diagram.

Gap at fracture in this case will be

$$g_2 = \frac{2xAE(\alpha t)^2}{2R^l} + L_0 \alpha t \text{ ————— (2)}$$

Where L_0 is the span length of the bridge provided with rail free fastenings.

Expressions (1) and (2) indicate that the gap at fracture is enhanced by an amount equal to $L_0 \alpha t$, when a girder bridge with rail free fastenings is located in the central portion of the LWR. Indian Railways have fixed the permissible gap at fracture as 50mm where by expression (2) becomes

$$2x \frac{AE(\alpha t)^2}{2R^l} + L_0 \alpha t < 50mm$$

This expression is applicable for both BG and MG tracks. However, as the wheel diameter of MG stock is smaller than

(73)

BG, the fracture gap of 50 mm is more critical for MG.

Over the years attempts have been made to increase the value of L_0 by adopting various techniques :-

(1) One way could be to increase the value of R, the longitudinal ballast resistance mobilized at the fracture. This could be done by :-

- Compacting the ballast in shoulders and cribs of the bridge approach sleepers.
- Enhancing the sleeper density to 1660 Nos./km in the bridge approach.
- Heaping up of ballast in the bridge approach starting from the foot of the rail.
- Box anchoring sleepers wherever required.

These measures have to be taken in the bridge approaches 50m on either side.

Table 1 of the LWR Manual 1996 gives the maximum overall length of girder permitted on LWR/CWR in with the following stipulations :

1. Girder bridge should have sliding bearings on each end with single span limited to 30.5m.
2. Rail should be provided with rail free fastenings throughout the length of the bridge from abutment to abutment.
3. The approach track should be suitably upgraded as mentioned above.

2) Another way of increasing the value of L_0 would be to improve the approaches as mentioned above in addition to providing a few sleepers on each span with creep resistant fastenings. The creep resistant fastening will hold the rail and prevent the gap at fracture from becoming excessive.

However, provision of creep resistant anchors implies an interplay of forces between the rail and grider. Hence the following stipulations are made for bridge provided with rail free fastenings and partly box-anchored (with single span not exceeding 30.5m and having sliding bearings at both ends).

(74)

- (1) On each span 4 central sleepers will be provided with creep resistant fastenings and remaining sleepers with rail free fastenings.
- (2) Bridge timbers laid on girders shall not be provided with through notch but shall be notched to accommodate the individual rivet heads.
- (3) The girders shall be centralized with reference to the location strips on the bearing before laying LWR/CWR.
- (4) The sliding bearings shall be inspected twice a year and oiling and greasing of the bearing carried out once in two years.

Table 1 of LWR Manual

Temperature Zone	Rail Section	Rail free fastening on bridge	Rail free fastening on bridge and partly box-anchored/ creep resistant fastenings
		Type of sleeper used in approaches	Type of sleeper used in approaches
		PRC/ST	PRC/ST
I	60kg	30m	77m
	52kg/90R	45m	90m
II	60kg	11m	42m
	52kg/90R	27m	58m
III	60kg	11m	28m
	52kg/90R	27m	43m
IV	60kg	11m	23m
	52kg/90R	27m	43m

The LWR Manual has also suggested some additional methods of carrying an LWR over bridges. These are discussed below :

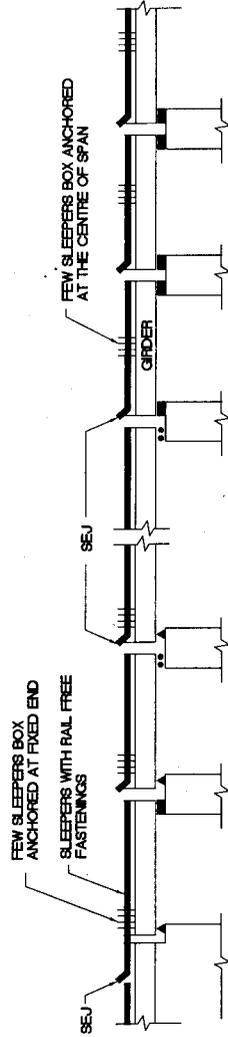
- (1) Providing an SEJ on each pier with rail free fastenings on the bridge. In order to avoid creep four sleepers on each span will be box-anchored. These sleepers will be at the fixed end of the girder, if the girder is having rollers at one and rockers on the other side. These sleepers will be at the centre of the span if the girders are having sliding bearings on both sides. This arrangement is shown in Fig.4.14.

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- (2) Providing an SEJ at the far end approach of the bridge using rail free fastenings over the girder bridge(Fig 4.15):
In this arrangement an SEJ is provided at the far end approach of the bridge(abutment away from the LWR) at a distance of 10m away from the abutment with rail free fastenings on the bridge proper.The SEJ will have to cater to the free expansion or contraction of the rail on the bridge as well as movement of the breathing length.Hence the SEJ will have to be a wide gap SEJ capable of accomodating larger movements.The permissible span lengths with normal SEJs and 190mm maximum gap SEJs are given on the adjoining page.

Rail temp. Zone	Max. movement of SEJ (mm)	Max. length of bridge with SEJ			Initial gap to be provided at t_d	
		with ST/ PRC approach sleepers	with CST-9 approach sleepers	with ST/ PRC approach sleepers	with CST-9 approach sleepers	
IV	190	55m	45m	7.0 cm	6.5 cm	
III	190	70m	70m	7.0 cm	6.5 cm	
II	190	110m	100m	6.5 cm	6.5 cm	
I	190	160m	150m	6.5 cm	6.0 cm	
II	120	20m	15m	4.0 cm	4.0 cm	
I	120	50m	50m	4.0 cm	4.0 cm	

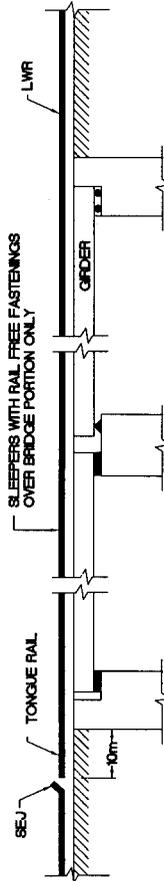
Note : SEJ is to be installed 10 metre away from the abutments.



NOTE:
TONGUE RAIL SHALL BE IN CONTINUATION
TO THE FREE END OF THE GIRDER

WELDED RAILS ON GIRDER BRIDGE (PIER TO PIER)

Fig. 4.14



LWR / CWR ON GIRDER BRIDGE WITH SEU AT THE FAR END
APPROACH OF THE BRIDGE

LEGEND

- ▲ ROCKER BEARING
- ROLLER BEARING
- SLIDING BEARING

Fig. 4.15

NOTE:
SEU TO BE INSTALLED 10 m AWAY FROM ABUTMENTS

CHAPTER V

LAYING AND MAINTENANCE

5.1 Laying of LWR

1. An initial survey of the section where the LWR/CWR is proposed to be laid should be carried out. A foot by foot survey is recommended.
2. Locations where LWR/CWR cannot be laid, need to be identified. These locations could be:
 - i) Sharp curves
 - ii) Bridge locations
 - iii) Steep Gradients
 - iv) Points and crossings
 - v) Troublesome formations
 - vi) Distressed bridges

These locations will be isolated from the LWR by providing SEJs on either side.

3. A detailed plan shall be made showing the exact location of the SEJs and various other features such as level crossings, yards, curves, points and crossings, gradients and bridges. This is called the LWR plan. This plan should be got approved by the T.H.O.D. if there are no deviations from the provisions of the LWR manual or by CE/CTE if any deviations are proposed.
4. Temperature Records : The LWR Manual (1996) prescribes that each railway should nominate 8 to 10 stations on its jurisdiction where temperature records over a period of 5 years should be built up by installing suitable continuous recording thermometers. The maximum and minimum rail temperature for a continuous period of at least 5 years shall be ascertained and the mean rail temperature for the region arrived at.

This could provide the basis for fixing the rail neutral temperature, ascertaining the periods during the year when

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maintenance operations could be carried out or hot weather and cold weather patrolling need to be introduced. If these records are not available, use may be made of the rail temperature map given in the LWR Manual.

5. Materials required for laying an LWR :
 - i) Four numbers of 6.5 metres or longer rail pieces of the same rail section as the LWR.
 - ii) 2 sets of SEJs with sleepers and fastenings.
 - iii) Adequate number of 1 meter long fishplates with screw clamps/joggled fish plates with slotted grooves and bolted clamps.
 - iv) Rail closures.
 - v) Rail cutting equipment.
 - vi) Welding equipment.
 - vii) Destressing equipment i.e. roller, tensor, wooden mallets.
6. Preliminary works to be carried out prior to laying the LWR :
 - i) Replacement of insulated joints by glued joints if LWRs are to be laid in station yards.
 - ii) Realignment of curves.
 - iii) Lifting or lowering of track to eliminate sags and humps.
 - iv) Introduction and improvement of vertical curves.
 - v) Stabilization of troublesome formation.
 - vi) Rehabilitation of weak bridges.
7. Welding of rails to form LWR :
 - i) Rails are supplied from workshop flash butt welding plants in 3-rail, 10-rail, 10-rail or 20-rail panels. Care should be taken while handling 90 UTS rails at the time of unloading.
 - ii) The 10-rail/15-rail/20-rail panels should be placed in the track and subsequently welded to form the LWR of required length.

- iii) Two complete sets of SEJs shall be inserted at predetermined locations with gaps in the mean position. Closure rails of 6.5 metres or longer length shall be provided on either side of SEJs to facilitate adjustment of gaps during destressing operation.
- iv) Laying of welded panels or welding of joints at site can be done at any time of the year. But after welding sufficiently long panels of about 1 km length or longer, destressing of the LWR should be undertaken as soon as possible.
- v) Speed restrictions (30 kmph) should be imposed if fishplated joints are existing in an LWR. If a temporary joint in the form of a 1m. long fishplate with screw clamps or a jogged fishplate/normal fishplate with screw clamps exists, then speed restrictions of 30 kmph and 20 kmph respectively should be imposed. A watchman should also be posted at a clamped joint.

5.2 Destressing Operations

5.2.1 One of the most important maintenance operations in the LWR is destressing the LWR. Destressing is the operation of removing the locked up stresses in the LWR and bringing the LWR to a stress-free state at a predecided temperature called the stressfree temperature. It is also called the neutral temperature. As per the LWR Manual, the stress-free temperature should lie between $t_m + 5^{\circ}\text{C}$ and $t_m + 10^{\circ}\text{C}$ for 60kg and 52kg rail sections and between t_m and $t_m + 5^{\circ}\text{C}$ for 90 R rail sections where t_m denotes the mean rail temperature.

5.2.2 Periodicity of destressing and conditions which would warrant destressing to be done :

Following the Khanna accident, a periodicity of once in 3 years for destressing every LWR had been prescribed. Subsequently these instructions were withdrawn and current instructions do not lay down any periodicity for destressing. The LWR Manual lays down that destressing should be done :

1. (i) when gap at SEJ goes beyond the prescribed limits.
(ii) when tongue rail/ stock rail cross the mean position.
2. After a special maintenance operation like deep screening.
3. After restoration of track following an unusual occurrence.
4. If the number of locations where temporary repairs have been done exceed 3 per kilometer.

It is suggested that the condition of the entire LWR should be considered before taking a decision of destressing.

5.2.3 Destressing operations could be done in two ways :

- (1) Manually
- (2) Using a tensor

1. Manual Destressing of the LWR : When the prevailing temperature t_p falls in the range prescribed for the stress-free temperature then manual destressing of the LWR can be resorted to. Work should be done in the presence of a PWI.

The steps involved are given below :

- A) Pre- Block Activities:
 - 1) All impediments to free rail movement such as check rails, rail anchors etc. should be removed.
 - 2) 4 Nos. of closure rails to be created at either end of the LWR, next to the SEJs.
 - 3) ERC clips should be greased so that their removal during the block will be easy.
 - 4) Prior to the block, 50% of the ERCs are loosened and a speed restriction of 30 kmph imposed.
- B) Block Activities:
 - 1) Arrange for block of adequate duration taking into account the length of the LWR to be destressed and the labour available.

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- 2) The closure rails are removed and placed on the cess at either end of the LWR.
- 3) ERCs are removed starting from the ends towards the centre. The rails are subsequently lifted off the rail seat and supported on rollers at every fifteenth sleeper. Side rollers should be used for curves.
- 4) The rails are tapped with a wooden mallet to remove any buildup stress and subsequently lowered on the rail seat after removing the rollers. This is a good opportunity to change the rubber pads.
- 5) ERCs are put back starting from the middle towards the ends. During this operation the temperature of the rail should lie within the prescribed range for the destressing or stress-free temperature.
- 6) The SEJs at either end are adjusted for a standard 40 mm gap at the destressing temperature. Suitable cuts in the closure rails can be made and inserted back in the track after which the block can be removed.
- 7) Welding of the closure rails can be performed if required in a separate block. Fig 5.1 shows schematically the operations involved in destressing manually with the options available.

2. Destressing using tensor :

When the prescribed temperature range for the neutral temperature is not available in the prevailing conditions, the rail tensor can be used for destressing operations. The rail tensor is a hydraulic or mechanical device which can create tensile stress in the rail by pulling the same.

Principle : The rail tensor creates tensile stress in the rail of such magnitude that these stresses completely balance the compressive stress created in the LWR when the temperature rises from t_p (the prevailing rail temperature) to t_n , the defined rail

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neutral temperature. The LWR is made stress-free at t_p by removing the fastenings and subsequently tensile stresses are created in it by pulling the LWR by a calculated amount :

Compressive stress created in the LWR due to rise of temp from t_p to t_n with zero stress at $t_p = E\alpha(t_n - t_p)$
The tensor creates the same amount of tensile stress by pulling the LWR section by an amount equal to Δ .

If the length of the section is L, then strain = $\frac{\Delta}{L}$

The tensile stress due to this strain will be $E \frac{\Delta}{L}$

$$\therefore E \frac{\Delta}{L} = E\alpha(t_n - t_p)$$

$$\text{or } \Delta = L\alpha(t_n - t_p)$$

This expression gives the extension to be imparted to a segment of length 'L' of the LWR to get the stress-free temperature at t_n .

Equipment : The tensor is basically a pulling device which could be hydraulic or mechanical. It is non-infringing, enabling trains to pass when in position. It grips the web of the rail using a special cam arrangement eliminating bending, dipped joints and rail-head marks. It can be dismantled into different parts for being transported, the weight of the heaviest component being 54 kg. (Details of a tensor are given in Fig 5.2) Some of the details of a tensor manufactured by CTR Industries, Pune are given below:

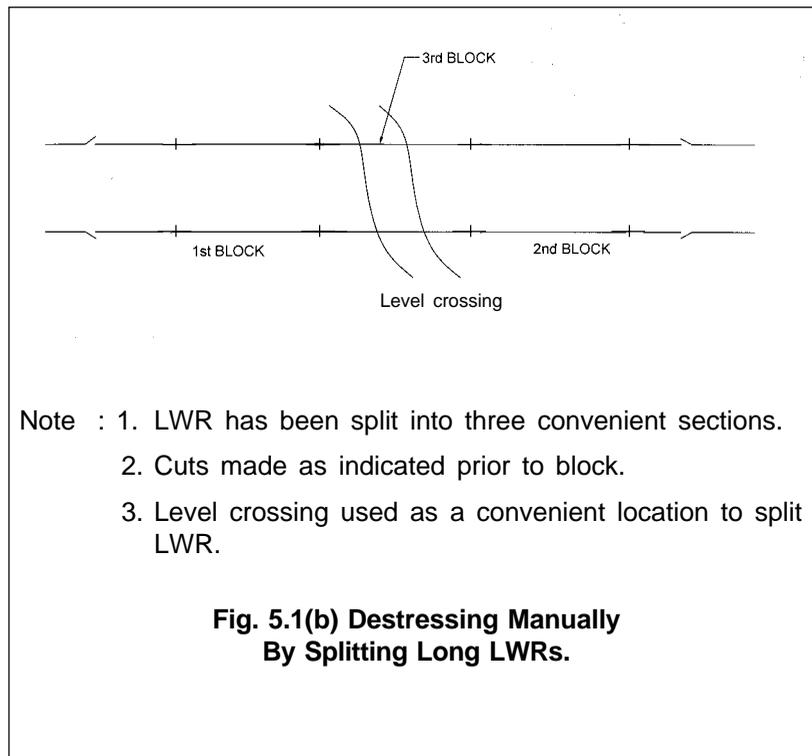
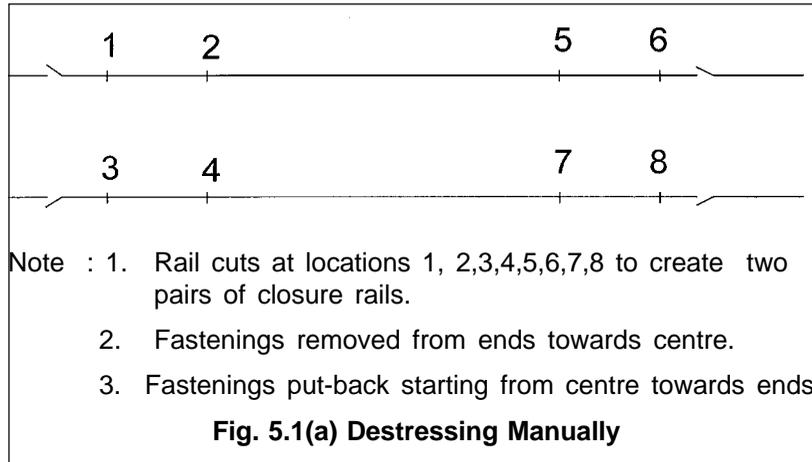
Total weight = 325 kg (without pump)

Pulling force = 70 tonnes

Pushing force = 30 tonnes

Stroke = 380 mm

Overall dimensions : 4500 mm length
1200 mm width



Steps in destressing by the rail tensor (Fig 5.3) :

- 1) The destressing operations have to be carried out when the prevailing rail temperature t_p is less than the the designed neutral temperature.
- 2) Make cuts 1m apart at the centre of the LWR.
- 3) Erect marker pillars W_0, W_1, W_2 at convenient distances of say 100m. The first marker pillar W_0 will be erected at a distance equal to the anchor length from the SEJ. The anchor length is the length of track where the fastenings will not be removed during the destressing operations. This length is required to resist the pull applied by the tensor. An estimate of the **anchor length** is as under:

For BG ..2.5 m per °C of $t_n - t_p$

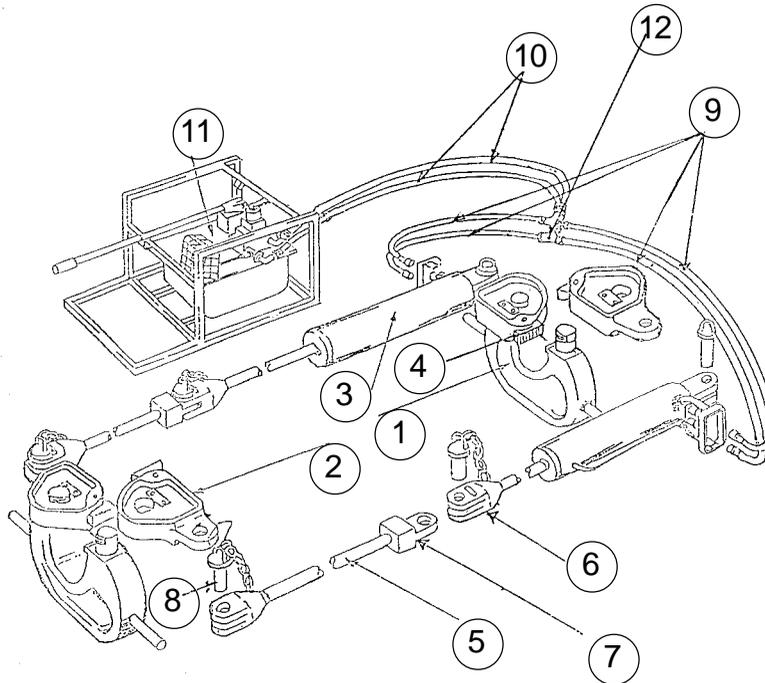
For MG ..4.5 m per °C of $t_n - t_p$

This anchor length is excluded from the destressing operations and will have to be done manually. The last marker pillar will be erected at the point where the LWR is being cut to make a gap of 1m. These marker pillars may be made on both the sides of the track. Transfer W_0 on to the foot of the rail.

- 4) Unfasten the fastenings starting from the end or from the centre where the rail is cut. Place the rail on rollers. Apply pull by tensor to get some movement at W_0 and release. Note movement at W_0 . If the movement is away from tensor, the zero correction is positive. Let it be Y_0
- 5) Transfer the remaining points W_1, W_2, \dots, W_a on to foot of rail and note down the prevailing rail temperature t_p . Calculate the extentions to be given in each segment. Extention to be given to first segment W_0W_1 will be,

$$= Y_0 + (W_0 W_1)x \alpha (t_n - t_p)$$

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PART LIST

Sr. No.	ITEM	QTY. Nos.	Sr. No.	ITEM	QTY. No.
1.	YOKE	2	7.	CLEVIS	2
2.	LEVER ARM	4	8.	PIN WITH CHAIN	6
3.	HYDRAULIC CYLINDER	2	9.	HOSE PIPE-2m	4
4.	JAW	4	10.	HOSE PIPE-4m	2
5.	TIE BAR	2	11.	HYDRAULIC PUMP UNIT	1
6.	FORK END	4	12.	T CONNECTOR	2

Fig. 5.2 HYDRAULIC RAIL TENSOR

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This distance is marked on the foot of the rail at W_1 away from the tensor.

Similarly, extension to be given to segment $W_1W_2 =$
Extension to segment $W_0W_1 + W_1W_2 \times \alpha \times (t_n - t_p)$

This distance is marked at W_2 away from the tensor.

- 6) Apply pull by tensor to get the required extension of segment W_0W_1 . This can be ensured by bringing the mark of required extension opposite to the mark on the marker pillar W_1 . Once the required extension has been given to a segment the segment can be lowered on to the rail seat and fastenings put back.
- 7) This process could be continued on to the other side of the tensor as well either simultaneously or after tackling one side of the LWR.
- 8) At the end in order to replace the 1 metre long closure rail at the centre make a paint mark at a distance of $(6.5\text{m} + 2 \text{ welds})$ measured from one rail end across the tensor. Remove the tensor and normalize the block.
- 9) In another block make a cut at the earlier made paint mark and put in a closure rail piece of 6.5m. One gap can be welded immediately. The other gap could be welded also, if 25mm, otherwise use the tensor to make the gap 25mm and do the welding.

5.3 Regular maintenance operations

5.3.1. Regular track maintenance operations as defined by the LWR Manual are :

- 1) Tamping/Packing
- 2) Lifting
- 3) Aligning including minor curve realignment
- 4) Shallow screening/ shoulder screening.
- 5) Renewal of fastenings requiring lifting of rail.
- 6) Maintenance of SEJ/buffer rails.

5.3.2. Most of the operations involve disturbance of the ballast bed leading to loss of the sleeper to ballast resistance. Consolidation of track is the process of building up of sleeper to ballast resistance either initially before laying of LWR or making up subsequent loss of resistance by any one of the following (Para 1.18 of LWR Manual)

- I
 - (i) For track structures consisting of sleepers other than concrete sleeper :
 - (a) Passage of at least 3,00,000 gross tons of traffic on BG or at least 1,00,000 gross tons of traffic on MG when compaction of ballast is done using hand-operated compactors/ consolidators or rammers.
 - (b) Passage of at least 50,000 gross tons of traffic on BG or at least 20,000 gross tons of traffic on MG or a period of 2 days whichever is later, when compaction is done by mechanized shoulder and crib compactor.
 - (ii) For track structure consisting of concrete sleepers :
Passage of 50,000 gross tons of traffic on BG or 20,000 gross tons of traffic on MG or a period of 2 days whichever is later.
- II One round of stabilization by Dynamic Track Stabiliser (DTS).
- III For newly laid LWR/CWR at least three rounds of packing, last two of which should be with on track tamping machines.

5.3.4 Temperature Restrictions while carrying out Maintenance Operations in LWR territory :

- (i) Normal maintenance operations should be carried out well before on set of summer. The temperature range within which such operations should be performed should be restricted to $t_d + 10^{\circ}\text{C}$ to $t_d - 30^{\circ}\text{C}$ where t_d is destressing temperature.
- (ii) If the rail temperature rises above $t_d + 20^{\circ}\text{C}$ during the period of consolidations which has been defined earlier (para 1.18 of LWR Manual) then the following steps have to be taken :

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- (A) For other than concrete sleeper track :
1. A speed restriction of 30 kmph in BG and 20 kmph in MG will have to be imposed if mechanical compaction of ballast has not been done during the maintenance operations in addition to posting of a mobile watchman.
 2. A speed restriction of 50 kmph in BG and 40 kmph in MG will have to be imposed if shoulder and crib compaction has been done during the maintenance operations.
- (B) For concrete sleeper track :
1. A speed restriction of 50 kmph in BG and 40 kmph in MG will have to be imposed.

5.3.5. Manual Through-packing of sleepers in LWR territory :

It must be remembered that till off track tampers are made available manual packing of concrete sleeper track with crowbars has been permitted (para 1408 of IRPWM). However, due to the necessity of opening out of ballast, for undertaking manual packing, the following precautions have to be observed:

- i) Only 30 sleepers should be opened out at a time, leaving the next 30 sleepers fully boxed and packed. The intervening sleepers can be opened out for packing after 24 hours if the GMT on the BG section is more than 10 and after 48 hours for BG sections with GMT less than 10 or in MG sections.
- ii) Only in case of emergencies and in the presence of PWI can 100 sleeper spaces be simultaneously opened out observing the usual temperature restrictions.

5.3.6. Other instructions while carrying out track maintenance in LWR territory:

- (i) Special attention shall be paid to

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maintenance of track at the following locations :

- SEJs/breathing lengths.
 - Approaches of level crossings, points & crossings and unballasted deck bridges.
 - Horizontal and vertical curves.
- (ii) Special attention shall be paid to maintenance of fastenings in LWR/CWR especially on concrete sleepers .
- (iii) Ballast section shall be properly maintained paying special attention to locations where the ballast profile could get disturbed such as at level crossing approaches, bridge approaches, curves, pedestrian and cattle crossings. In order to prevent rolling of ballast down the slope, the cess should be properly maintained with regular cess repairs. Dwarf walls should be provided wherever track trespass is anticipated to prevent loss of ballast.
- (iv) Adequate ballast should be arranged before going in for any maintenance operation such as lifting. Ballast should be recouped well before the onset of summer. Ballast procurement being a long lead item, adequate planning should be made for timely procurement of this vital item.
- (v) While slewing of track using crowbars, care should be taken to avoid simultaneous lifting of track.

5.3.7 Mechanised Track Maintenance :

- (i) Maintenance tamping using track machines can be done in continuation from one end of the section to the other. General lift should not exceed 50 mm for concrete sleepers and 25 mm for other than concrete sleepers.

Rail temperature restrictions are the same as laid down for manual track maintenance.

- (ii) Lifting of track where needed, in excess of 50 mm for concrete sleepers and 25 mm for other sleepers shall be carried out in stages with adequate time gap in between the successive stages, such that full consolidation of the previous stage is achieved prior to taking up the subsequent lift.

5.3.8 Casual renewal of sleepers, rails and fastenings :

The following precautions should be observed :

- (i) Casual renewal of sleepers : Not more than one sleeper in 30 sleepers shall be replaced at a time. Should it be necessary to renew two or more consecutive sleepers in the same length, they may be renewed one at a time after packing the sleeper renewed earlier duly observing the temperature limits.
- (ii) Casual renewal of fastenings :
 - a. When fastening renewal does not require the rail to be lifted, fastenings of not more than one sleeper at a time shall be renewed at a time, while at least 15 sleepers in between shall be kept intact. Work shall be done under the supervision of a keyman.
 - b. Fastening renewal requiring lifting of the rail such as replacement of the grooved rubber pads shall be done in the presence of the gang mate with at least 30 sleepers in between to be kept intact.

5.3.9. Maintenance of SEJs and buffer rails :

- i) SEJs should be checked, packed and aligned once in a fortnight. Oiling and greasing of the

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tongue rail and stock rail should be done simultaneously.

- ii) Buffer rails are free rails placed in lieu of Switch Expansion Joints.
 - (a) In rail temperature Zone I and II, 3 Buffer rail pairs, while in zone III and IV, 4 buffer rail pairs shall be provided. On BG, the buffer rail shall be 6.5 m long while for MG it will be 6.0 meters.
 - (b) Buffer rail joints are lubricated twice a year when the rail temperature is between $t_d + 15^\circ\text{C}$ and $t_d - 15^\circ\text{C}$. A standard gap of 7.5 mm is provided at a buffer rail joint at temperature t_d .
 - (c) In zone III and IV, if the gap closes at a temperature lower than $t_d + 30^\circ\text{C}$ or opens out to 15 mm at a temperature higher than $t_d - 30^\circ\text{C}$ it would indicate :
 - i) Defective initial gaps.
 - ii) Inadequate packing in breathing length.
 - iii) Creep of LWR.
 - iv) Movement of rail over sleeper in breathing length.

In zone I & II the lower temperature limit for gap fully opening out and the upper temperature limit for gap fully closing shall be taken as $t_d - 25^\circ\text{C}$ and $t_d + 25^\circ\text{C}$ respectively.

In such cases, the LWR should be destressed and the gap at the buffer rail joints restored to 7.5 mm at t_d .

5.4 Special Track Maintenance :

These works are generally carried out with a speed restriction in force.

This would include the following items :

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- i) Through fitting renewal
- ii) Deep screening/ Mechanised cleaning of ballast.
- iii) Major lowering/ lifting of track
- iv) Major realignment of curves.
- v) Sleeper renewal other than casual renewal.
- vi) Formation rehabilitation.

5.5. Deep screening in LWR territory :

Provisions laid down in Para 238 of IRPWM will also apply *mutatis mutandis* to LWR/CWR. This implies that

1. Work will be done with an SR of 20 kmph in the presence of a PWI.
2. While tackling two sleepers simultaneously, there should be at least 4 intermediate sleepers fully packed and boxed.
3. Deep screening could be carried out in continuation from one end of the section to the other in the above manner.

The temperature restrictions are as under :

1. Work should normally be done in the rail temperature range of $t_d - 20^{\circ}\text{C}$ to $t_d + 10^{\circ}\text{C}$.
2. If there is a possibility of rail temperature rising above $t_d + 10^{\circ}\text{C}$ during the execution of work then temporary destressing at a temperature 10°C below the anticipated maximum temperature should be carried out. This will keep the track in the safe zone as far as development of high compressive forces is concerned. If the temperature were to fall by more than 30°C below the temporary destressing temperature, cold weather patrolling should be introduced.
3. Temporary destressing should be done again after 15 days if there is wide fluctuation of temperature and there is a possibility of temperature rising further beyond the anticipated maximum temperature.

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4. Once the deep screening is completed, then the entire LWR should be destressed to bring the destressing temperature to the normal range.

CHAPTER VI

UNUSUAL OCCURENCES IN LWR, INSPECTION & RECORD KEEPING

6.1 Introduction: This chapter is devoted to a discussion on various unusual occurrences which may occur in an LWR such as fractures and buckling, and remedial measures to be taken in the event of the same.

6.2 Fractures

6.2.1 Rail and weld fractures occur with increasing frequency on LWR sections in winter due to development of longitudinal tensile forces as rail temperatures fall below the distressing temperature. The increasing incidence of rail fractures in LWR sections could also be attributed to the fact that the rail distressing temperature has been fixed between $t_m + 5^{\circ}\text{C}$ to $t_m + 10^{\circ}\text{C}$ for 52 kg and 60 kg rails increasing the tensile force created in the rail as temperature drops towards t_{min} .

Causes of fracture : Apart from excessive tensile thermal forces which could arise in a rail in LWR sections, fractures could occur in a rail due to variety of causes :

1. Material defects originating during the manufacturing process such as clusters of non-metallic inclusions, hydrogen flakes, rolling marks, guide marks etc. which may be present in spite of the non-destructive tests carried out on the rails during their quality assurance examination.
2. Residual stresses induced during manufacture : cooling, rolling, straightening etc.
3. Defects occurring due to incorrect handling of rails e.g. plastic deformation, scoring, denting etc.
4. Defects associated with faulty welding.
5. Dynamic stresses caused by vertical and lateral loads particularly by vehicles with wheel flats or when the vehicle runs over poorly maintained rail joints etc.

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6. Fractures due to corrosion at rail seat and liner location etc.
7. As discussed earlier excessive tensile forces in the rail generated due to temperature changes of the LWR.

6.2.2 Repairs to Fractures

1. Once a fracture has occurred, the railway official detecting the fracture should take immediate steps to block the section and prevent any train movement over the fractured portion.
2. This done, fracture repairs are done in the following stages :

A. Emergency Repairs

These repairs are carried out to pass the train over the fractured rail. The following steps are involved :

1. If the gap at the fracture is less than 30 mm, a fishplate for a rail fracture or a joggled fishplate for a weld fracture is fixed, using 4 tight screw clamps without a rail closure piece.
2. If necessary, a wooden block may be inserted below the rail to support the fractured joint.
3. If the gap at fracture is more than 30 mm, a rail-closure piece will be inserted into the gap after which fishplates or joggled fishplates will be fixed.
4. This done, the train is allowed to move over the joint with a SR of stop dead and 10 kmph for the first train and 20 kmph for the subsequent trains. The LWR Manual authorises a keyman / gangman to pass the train in such an emergency. If one meter long fish plates are used during the repair the SR will be 30 kmph.

B. Temporary Repairs

This essentially involves the removal of the fractured rail from the track and replacing it with a sound rail-closure piece of length generally more than 6.5 m.

The following steps are involved in temporary repairs.

1. Two paint marks are made on either side of the fractured

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joint at a distance, say X and Y as shown in the Fig 6.1.

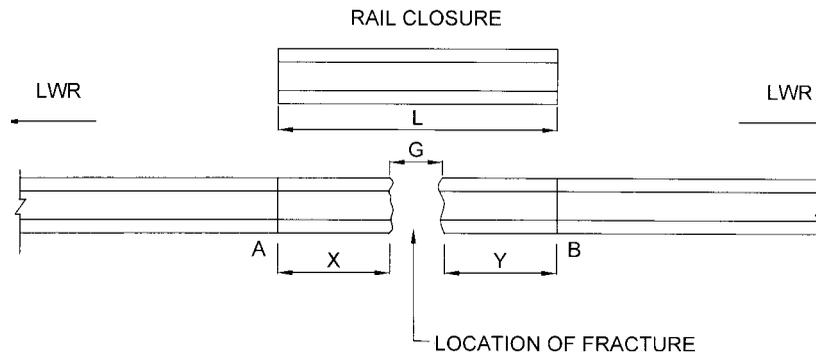


Fig 6.1

2. These distances X and Y are correlated to the length of the closure rail piece which is to be inserted into the track. Let the length of this closure rail piece be L. While replacing the fractured rail, the principle to be observed is that the rail inserted into the track should be equal to the length of the rail removed.

Here length of rail inserted = $L + 2 \text{ welds (50mm)}$

Length of rail removed = $X + Y + 2 \text{ saw cuts (say 1mm)}$

Hence $L + 2 \text{ welds (50 mm)} = X + Y + 2 \text{ saw cuts (say 1mm)}$

This relationship enables fixing up of paint marks on either side of the fractured rail joint at distances of X and Y from the fracture location.

3. During a block of adequate duration, rail cuts are made on either side of the fractured joint at the paint marks made earlier. The closure rail piece of length 'L' is inserted into the gap created and fishplates with screw clamps fixed at the two joints.

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4. In the same block if time permits, or in another block one of the gaps is adjusted to 25 mm (for SKV welding) and welding performed.

If the other gap is also 25 mm then the other joint could also be welded. However, if the other gap is not 25 mm, as will generally be the case, then the other gap is fishplated with screw clamps and opened to traffic.

5. For welding the other gap, a tensor is used. It will be used for reducing the gap to 25 mm. This is done by removing sleeper fastenings over a given length and applying the requisite pull by tensor. With the tensor in position and gripping the rail web, the rail joint is welded. The tensor is kept in position till weld metal cools down. After the weld metal has cooled down, the tensor is removed, and fastenings removed over a length of 125 m on either side of the weld. The rail is tapped to equalise the stresses and fastenings put back in position.
6. To summarise, the process of temporary repairs is carried out without adding any additional rail metal to the LWR. If the gap at fracture is bridged by providing a longer length closure rail, there will be a drop in the neutral temperature, creating high compressive stress in the rail during the summer season. The temporary repairs should be carried out in the supervision of a PW Mistry/PWI.
7. A new development in welding technology is wide gap welding. This enables fracture repairs to be done by providing a single weld instead of two welds as was done earlier. The method of repairs has been indicated in Fig. 6.2.

A,B are paint marks on either side of fracture.

Rail inserted = 75mm. (wide gap weld)

Rail removed = $X+Y+2$ saw cuts (1mm)

$$75 = X+Y+2 \text{ saw cuts}$$

(100)

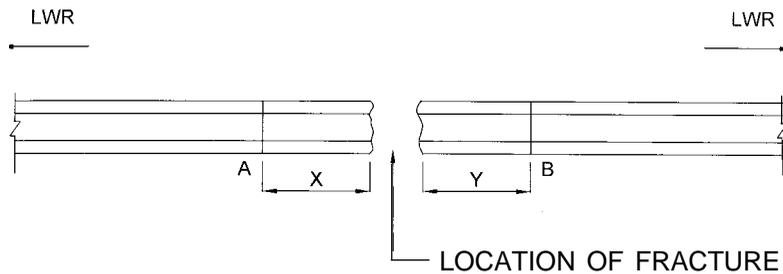


Fig. 6.2

C. Permanent Repairs :

This will involve destressing the entire LWR after a number of fractures have occurred in the LWR. The Manual prescribes destressing to be done when the number of fractures exceeds 3 per km.

D. Equipments required for fracture repairs :

1. Fishplates/ Joggled fishplates with bolted clamps.
2. One metre long fishplates with bolted clamps.
3. Rail Closure Pieces with different lengths.
4. Welding equipment with all accessories.
5. Tensor for obtaining the standard gap.

6.3 BUCKLING :

6.3.1 Buckling is the phenomena describing the sudden lateral shift in the alignment of an LWR to relieve the built-up compressive forces during the summer months as the temperature rises above the destressing temperature. Buckling results in complete distortion of the track geometry affecting safety and it is not possible to pass a train over the buckled track. Over the years, lateral stability of LWR track in hot weather conditions has been a source of great concern to track engineers. Initially it was thought that the slender rail section would not be able to take the high compressive forces generated during the summer season. Subsequent investigations by

various railways, however, indicate that the track strength against buckling was contributed by not only the rails, but the rail sleeper assembly with fastenings, and the ballast contributed in a substantial measure to the strength against buckling.

6.3.2 Some of the factors which could lead to buckling are :

1. Non-observance of the specified temperature restrictions while performing maintenance operations in an LWR.
2. Lack of ballast affecting the lateral and longitudinal ballast resistances.
3. Missing fittings.
4. Settling formation resulting in poor alignment of track.
5. Improper functioning of the SEJ.

6.3.3 Steps to be taken to avoid buckling :

1. Ensuring proper ballast profile.
2. Full complement of fastenings and anchors.
3. Observance of specified temperature restrictions ($t_p \leq t_d + 10^\circ\text{C}$) during maintenance operations.
4. Introduction of hot weather patrolling when the prevailing rail temperature goes beyond $t_d + 20^\circ\text{C}$, ($t_p > t_d + 20^\circ\text{C}$).
5. Controlling misalignments in track.
6. Keeping a close watch on SEJ gaps specially during extreme temperatures.
7. Proper repairs of fractures i.e. avoiding addition of metal at the fracture location during repairs.

6.3.4 Steps to be taken in face of impending buckling :

On detecting severe sunkinks or noticing hollowness of sleepers as detected by a canne-boule, the following steps are suggested :

1. The section to be blocked or speed restrictions to be imposed, depending upon the severity of the situation.
2. Additional ballast to be dumped on the shoulder, by taking out, if required, ballast from the centre of the track.

3. At tight rail locations, rail should be cut out from the track. In the morning times, this could be achieved using a hacksaw blade. However, this will not be possible when the rail is under compression as it will tend to pinch the blade. Gas cutting to cut rail out will have to be resorted to. Subsequently the heat affected martensite zones could be removed by cutting three inches on either side of the gas cut by a hacksaw blade. The golden principle to be followed is : **“When in doubt, cut rail out.”**

6.3.5 Repairs to be undertaken in the event of buckling :

1. Each case of buckling shall be investigated by the AEN soon after its occurrence and a detailed report submitted to the DEN/Sr DEN.
2. The rectification shall normally be carried out in the following stages under supervision of the PWI.

(A) Emergency Repairs

1. This repair is carried out to restore traffic on the section. A 6.5 m long rail piece will be cut out from the buckled track at the location of buckling, resorting to gas-cutting if required.
2. After removal of the rail piece, it will be possible to slew the track back into proper alignment.
3. A closure rail piece of suitable length could now be inserted into the track and section restored after fixing fishplates and screw-clamps.

(B) Permanent Repairs

The clamped closure rail piece will be welded at either end. To get the required gaps for welding, rail cutting equipment will be required. In order to complete the repairs, destressing of the entire LWR will be carried out as early as possible.

6.4 Inspection of the LWR and Record keeping :

6.4.1 Inspection: While an LWR section, reduces the maintenance requirements it necessitates intensive inspections at supervisory and officers' level. The Sr. DEN / DEN, AEN & PWI and other inspecting officials should pay special attention to the aspects given below while inspecting LWR sections.

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1. Ballast adequacy and maintenance of stipulated ballast profile specially at locations where the profile is likely to be disturbed due to trespass.
2. Special attention at vulnerable locations such as curves, level crossings, girder bridge approaches, etc.
3. Knowledge of rules and regulations (specially temperature restrictions) of Mates, Keymen, Gangmen and P Way Mistries for track maintenance in LWR territory.
4. Action to be taken by P. Way Mistry, Mate, Keyman and Gangman in the event of a fracture or buckle.
5. Ultrasonic Flaw Detection of rails and welds should not be in arrears.
6. Inspection of SEJ gaps and creep movement in central portion of LWR / CWR should be as per schedule given below:
 - (i) PWI / APWI - To measure the SEJ gaps alternately once in 15 days during the two hottest and two coldest months of the year. In the remaining 8 months, they will measure at 2 monthly intervals again alternately.
 - (ii) The sectional AEN will measure the SEJ gaps once in 6 months preferably during the coldest and hottest months.

6.4.2 Records:

- (1) The PWI should maintain a permanent register called the LWR section register. This register should record various details of the LWR as laid down in annexure XI and XII of the LWR Manual.
- (2) An indication plate should be fixed on the cess at each SEJ, showing the date of distressing, distressing

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- temperature and length of LWR / CWR.
- (3) Inspection of SEJ gaps and creep movement in the central portion will be recorded as per laid down frequency in the proforma prescribed in Annexure- XIII(A) and XIII(B).
 - (4) PWI / AEN / DEN will carefully study the SEJ gaps and creep in the central portion before deciding the remedial measures required to be taken.
 - (5) AEN will analyse the observation of each LWR / CWR in his jurisdiction and give a certificate at the end of the LWR / CWR section register before onset of summer regarding satisfactory behaviour of all LWRs / CWRs on his section. DEN / Sr.DEN will scrutinize observations of each LWR /CWR, initial each page and send exception report to Territorial Chief Engineer for his decision / orders.

6.5 Duties, responsibilities and staff training

Readers may refer to para 9 of LWR Manual 1996 to know more about the duties, responsibilities and training of staff for working in LWR territory.

CHAPTER VII

SPECIAL TOPICS

7.1 Buckling Phenomena : As described earlier, buckling is the sudden lateral shift in the track alignment to release the built up compressive forces in the rail. The strength of track against buckling or what is described as lateral stability of track has been investigated in great detail by various railways. The studies conducted by various railways and the results thereof have been discussed in this chapter.

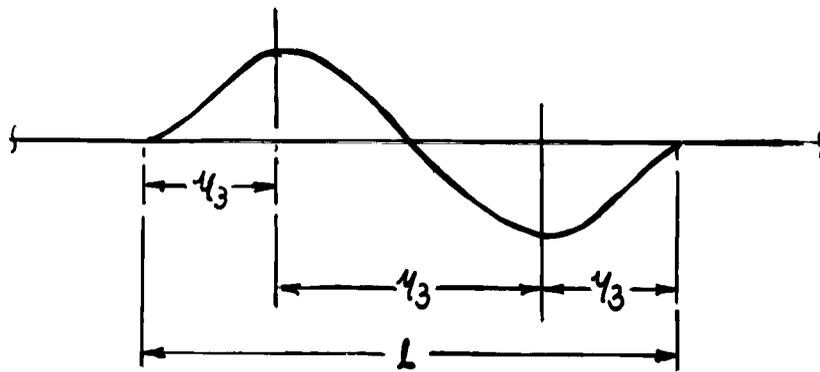
7.2 Tests by German Railways: Results of a series of track buckling tests conducted for the Federal German Railways were reported by F. Birmann and F. Raab in 1960. The test facility was located at the Technical University of Karlsruhe. The track section was 46.50m and was confined at both ends by reinforced concrete blocks. The following results were obtained from the tests :

- 1) In all the tests the track buckled laterally. The buckling modes exhibited 2,3 or 4 noticeable half waves each of length 5 to 6 metres. The largest amplitude of displacements was about 25 centimetres. This implied that a buckled track could have several shapes with buckling taking place in several wave forms.(Fig 7.1) Buckling in the form of a 'C' could occur on a sharp curve (First wave form) while buckled track resembling an 'S' shaped curve is generally evidenced on straight tracks (2nd wave form). The force diagram after a buckle is shown in Fig 7.2. It indicates that while a track physically buckles over a length 'l' the force diagram is affected over a length 'a' where 'a' is several times 'l'.
- 2) Straight tracks with smaller lateral imperfections buckled at much higher temperature increases than those tracks with noticeable lateral imperfections. Buckling of straight tracks occurred suddenly with a loud bang (explosive buckling) while the imperfect track buckled gradually and quietly(passive buckling).

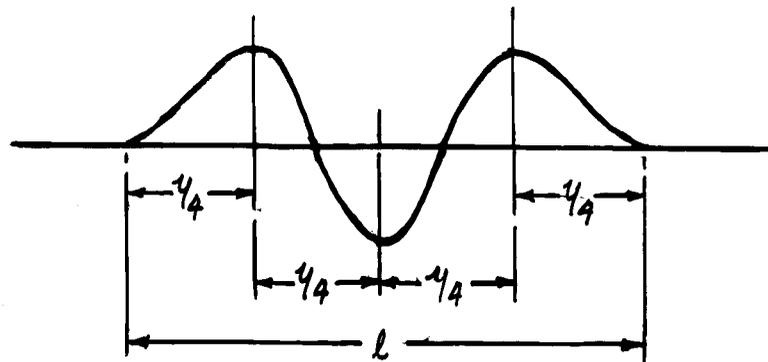
(106)



First waveform



Second waveform



Third waveform

Fig. 7.1

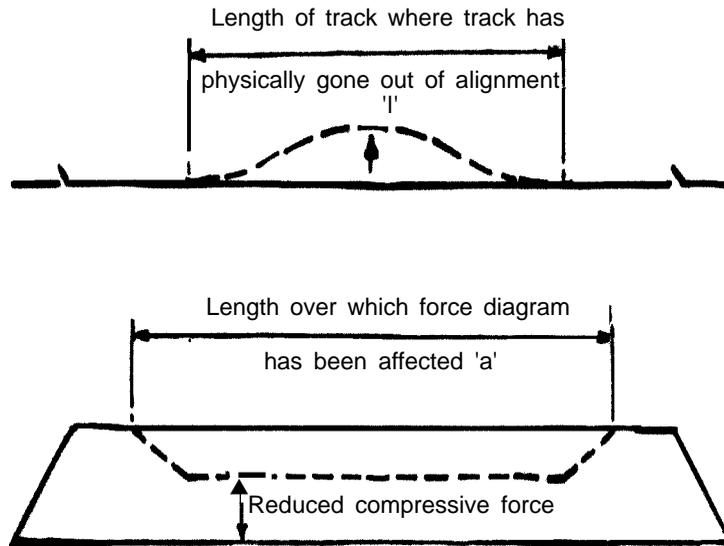


Fig. 7.2

- 3) With use of different fasteners, the buckling load varied by as much as 25%
- 4) Over a period of time with reversal of temperatures there is an accumulation of undesirable permanent lateral track deformations for temperature increases which do not cause actual track buckling but definitely increase the buckleproneness. This is shown in Fig. 7.3.

7.3 Studies Conducted by British Transport Commission : In order to study the conditions and factors affecting the stability of the Long Welded Rails a large testing program was started in 1953 by the Civil Engineering Laboratory of the Western Region of British Railways. These researches were carried out and described by Mr D.L.Bartlett, Assistant Director of Research (Engineering), Research Department, British Railways.

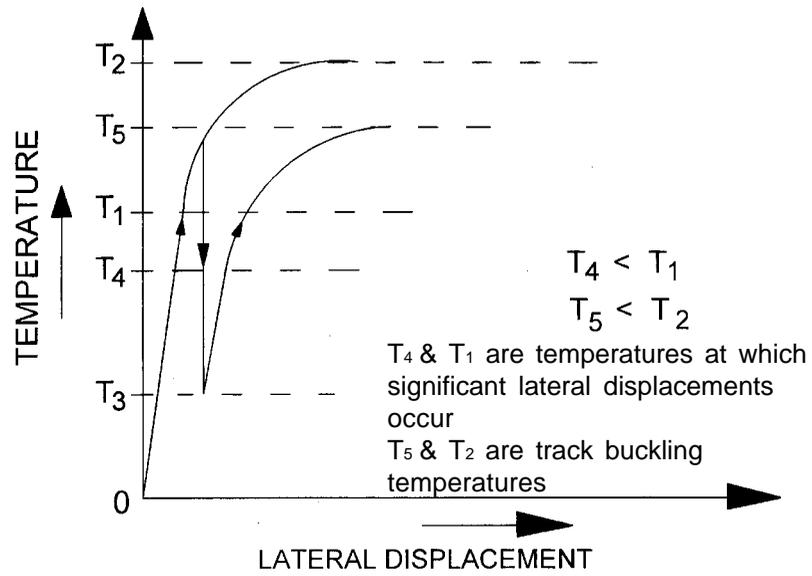
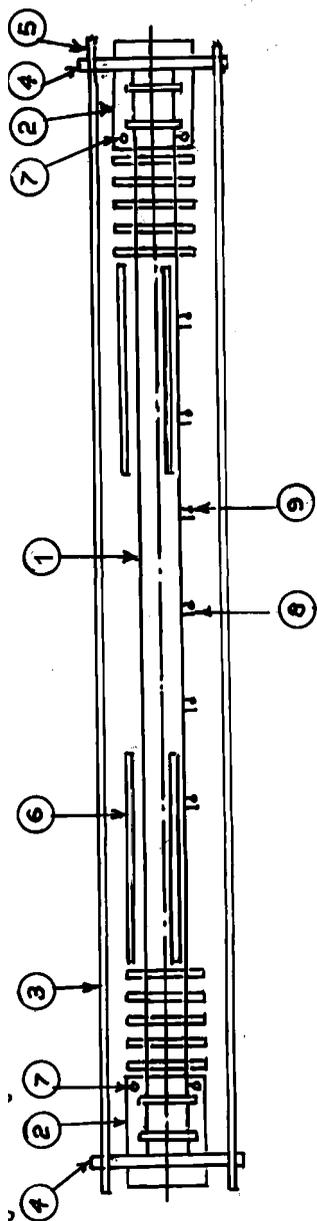


Fig. 7.3 ACCUMULATION OF LATERAL DISPLACEMENTS



- 1. 120 ft length of track
- 2. End anchorage blocks
- 3. Tie bars
- 4. Restraining beams
- 5. Hydraulic jacks
- 6. Electric heaters
- 7. Dial gauges registering longitudinal rail movement
- 8. Thermometers
- 9. Dial gauges registering lateral rail movement

Fig. 7.4

7.3.1 Test Arrangement (Fig. 7.4)

The main tests devised for the purpose of carrying out buckling tests was a 120 feet test bed upon which could be built, complete in every respect a length of track, the whole capable of being subject to thermal stresses. The arrangement of the test bed was such as to simulate the central portion of a long welded rail length on site which does not move longitudinally with temperature change. The test bed was laid inside a disused tunnel where a constant ambient temperature could be expected.

The 120 feet track rails were anchored at each end to concrete blocks sunk below ground level. This was sufficient to prevent rotation of the track and change of gauge but not to prevent the expansion of the rails. The latter was controlled by four tie bars, two on each side of and clear of the test track. Any tendency for the rails to expand could be counteracted by the jacks, although it must be stressed that the jacks were not directly used to induce compression in the rails. Four dial gauges attached to an independent datum registered any longitudinal movement of each rail end during the tests. By operating the jacks the rail lengths could be kept sufficiently close to their original values to be consistent with actual conditions in the field.

Heaters : Electric heaters with parabolic reflectors were used to simulate the heat radiation from the sun; they were situated on one side of each rail at a distance determined experimentally so that the rate of heating was not excessive.

Thermometer : Normal glass and mercury thermometers inserted in sockets drilled mainly in the head of the rail were used to measure the temperature.

Misalignment : This is the offset of the rail from the straight. The length of misalignment is the length over which misalignment occurs. The track was laid initially as straight as possible and then given a small misalignment over a given length.

Methodology of Test

Using the above setup, the longitudinal load required to buckle the track was determined experimentally for different types of sleepers,

fastenings and ballast packing conditions. Using theoretical methods the longitudinal load required to buckle a track was determined and the same compared with experimental values.

7.3.2 Buckling Load Formula:

The formula derived for the longitudinal load required to buckle a straight track is :

$$P = \frac{\Pi^2 E I_s}{L^2} + \frac{\Pi^2 C}{16D} \sqrt{\frac{\Pi L}{q}} + \frac{W_{\max} L^2}{\Pi^2 q}$$

Where

I_s is the moment of inertia of the two rails put together in the horizontal plane.

L is the distance between the points of contraflexure of the buckled track.

C is the torsional coefficient for the given type of fastening

$T = C\sqrt{\alpha}$, Where T is the torque resisting buckling and

α is the angle of twist for the fastening due to rotation of the rail on the rail seat

D is sleeper spacing

q is the misalignment of the track over length L

if W_{\max} is the lateral ballast resistance per meter length of track

and W is the lateral ballast resistance per sleeper then

$W_{\max} = W/D$

Analysing the above expression, it can be seen that :

1) $\frac{\Pi^2 W I_s}{L^2}$ represents the contribution of the rails to resistance

against buckling. Little can be done to this term, as it is dependent mainly on the properties of the rail.

2) $\frac{\Pi^2 C}{16D} \sqrt{\frac{\Pi L}{q}}$ represents the contribution of the sleeper/fastening

combination to the resistance against buckling. Here clearly a reduction in sleeper spacing D or an increase in the fastening torsional co-efficient C will cause an increase in the overall resistance to buckling

(112)

3) $\frac{W_{\max} L^2}{\Pi^2 q}$ represents the contribution of the lateral ballast resistance.

The following points are to be noted:

1. If the track were perfectly straight and points of equal load application central for each rail, then the track would not buckle however great the longitudinal compressive force. However, in practice no track exists under these ideal conditions and a misalignment of 'q' over a length 'L' will always be present. In any case, it is evident that the lower the L /q ratio, the smaller will be the buckling load. It means that large misalignments significantly reduce the strength against buckling.
2. Experimentally it has been observed that when a buckling occurs, the sleepers remain at right angles to the original track alignment. For this to occur, the rail must rotate on the rail seat. Clearly, only one thing resists such a rotational movement and this is the torsional resistance (denoted by torsional co-efficient C) afforded by the fastenings. Clearly the buckling load is proportional to torsional resistance.
3. 'L' the length of buckled track is taken as 20 feet for all cases. In actual fact for a given combination of C,D, Wmax and q there exists only one value of 'L' which will yield a minimum value of 'P' (the buckling load). Hence for various combinations of these variables, a range of 'L' values would emerge. For practical use however, 'L' is chosen as 20 feet and the value of 'q' as 1/4 inch(6mm).
4. The relative contributions of rails, rail sleeper fastenings. and ballast would depend upon the actual conditions prevailing at site. Under normal conditions the percentage contributions could be 10%,30% and 60% respectively.
5. The buckling load values as determined experimentally show a fair correspondance (within a few per cent) with the values determined from theoretical calculations.

6.A PWI can ensure that the track remains safe against buckling by:

1. Reducing the lateral misalignment in the track.
2. Ensuring that no sleeper rail fastenings are missing.
3. Providing full complement of ballast in the track as per prescribed ballast profile.

7.4 Static Buckling and Dynamic Buckling

The discussion so far has been centred on buckling caused by longitudinal compressive force buildup due to rise of temperature above the stress-free temperature. This buckling due to thermal loads alone is called static buckling. The industry today is more concerned with buckling caused by the movement of a train on the track in the presence of thermal loads. Such a buckling is called dynamic buckling. The effects of a moving train which could contribute to dynamic buckling are as given below :

- 1) Loaded axles of a moving train cause the track to be lifted in front of, in the rear of or even between the moving axles. The wave so created as seen in the vertical profile of the rail in front of the engine is called the precession wave, in the rear the recession wave and in between the axles, the central wave. Any of these waves

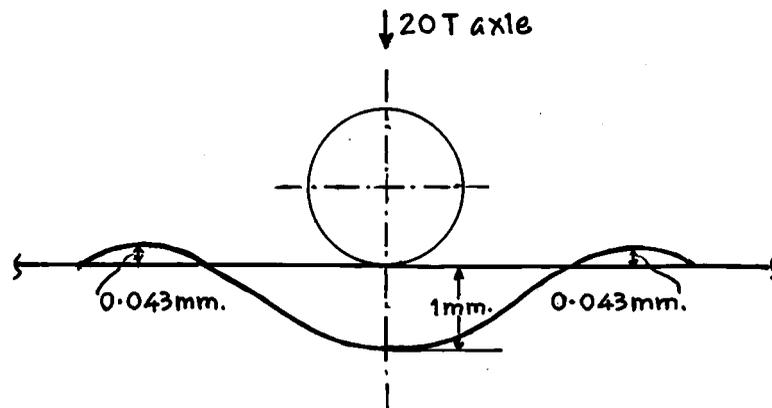
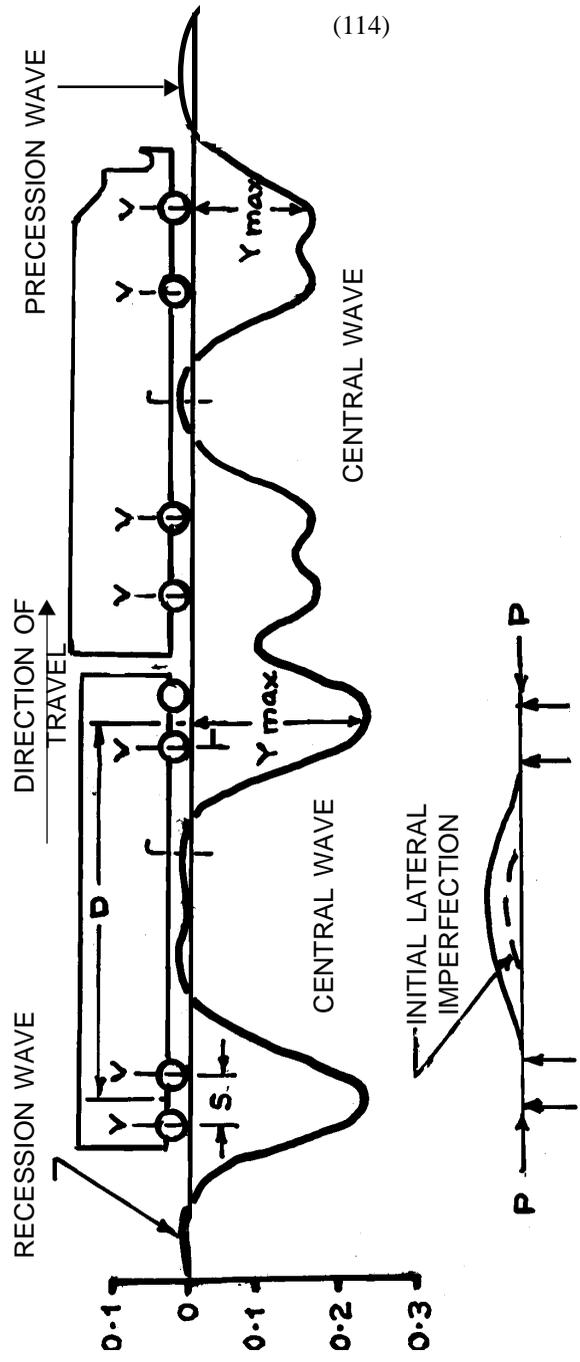


Fig. 7.5 (a)



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ASSUMED LATERAL BUCKLING MODE

Fig. 7.5 (b)

could be critical enough to cause loss of contact between the ballast and the sleeper soffit resulting in the loss of lateral ballast resistance thereby making the track buckleprone (Fig7.5(a)&(b)).

- 2) Tractive and braking forces applied by the moving train change the force level in the LWR and continuous braking at a given location could result in buildup of compressive forces creating buckling tendencies in the rail.
- 3) The hunting motion of the moving train over lateral misalignments in the track could create large lateral forces producing buckling tendencies.
- 4) Vibrations induced by the moving train could disturb the ballast and lower the lateral ballast resistance.

7.5 Dynamic Track Buckling Model:

The effect of a moving train increasing tendency of a track to buckle when the temperatures are rising or the response of the track to disturbing lateral forces is depicted by what is called DYNAMIC TRACK BUCKLING MODEL. This model is essentially a relationship between the lateral track displacement and the temperature increase over the force free or neutral temperature. The model is depicted in the figure (Fig7.6).

The model has 3 limbs as shown: AB is the prebuckling limb while

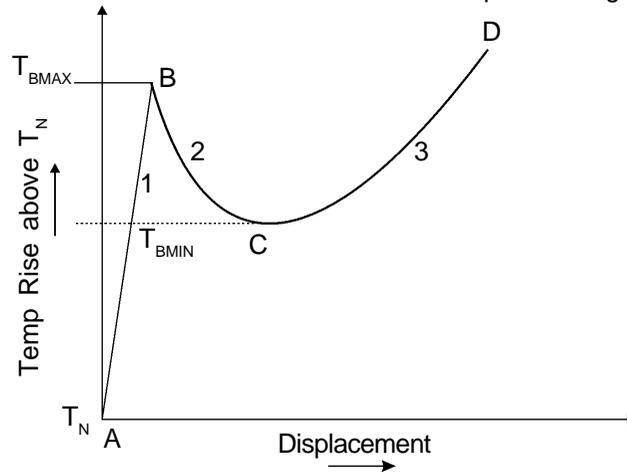


Fig. 7.6 DYNAMIC TRACK BUCKLING MODEL

BC and CD are post-buckling limbs. At B when the temp. rises to T_{BMAX} , the track becomes unstable where even an infinitesimal lateral force will cause the track to buckle. Below temp T_{BMIN} at point C, even a large force will not be able to buckle the track. Between points 'B' and 'C' a moving train could impart sufficient force to buckle the track. Between 'B' and 'C' the track on buckling will first move to an unstable buckled phase on curve BC and subsequently to a stable buckled phase on CD. It is assumed that if the track can be brought into position 2 it will automatically move into position 3. At T_{BMAX} , the energy required to buckle a track is almost zero while below T_{BMIN} the energy required to buckle a track is much larger than that which could be provided by a moving train. Between T_{BMAX} and T_{BMIN} , the transition from the pre-buckling stage to the unstable buckled state and to the stable buckled state could be effected under the influence of energy imparted by the moving vehicle.

Various softwares have been developed to predict the T_{BMAX} and T_{BMIN} temperatures for given track and rolling stock parameters. In the USA, the program developed is called CWR-BUCKLE. Other software programs are CWRSAFE and related programs. The inputs to these programs are :

- 1) Rail section
- 2) Track curvature
- 3) Rolling stock characteristics
- 4) Lateral ballast resistance
- 5) The misalignment in the track.

Using these programs the T_{BMIN} and T_{BMAX} of the given track for a given set of parameters are determined. The policies regarding LWR maintenance can then be decided. These could include :

- 1) Allowable Temperature rise above t_n for maintenance activities
- 2) Temperature at which track enters the danger zone, and necessitates hot weather patrolling.

7.6 CWR Safety Assurance Program:

The discussion above could form the basis of a continuous welded track buckling safety assurance program. For safe operations of CWR track with respect to buckling, the allowable temperature increase T_{ALL} over the neutral temperature should be greater than the

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difference between the maximum anticipated rail temperature T_{MAX} on a given day and the neutral or stress-free temperature T_n i.e.
 $T_{ALL} > (T_{MAX} - T_n)$

The expression on the right is the anticipated rail temperature rise over T_n while T_{ALL} is the allowable temperature rise which could be determined from the values of T_{BMIN} and T_{BMAX} . T_{ALL} will be somewhere between these two extreme temperature values depending upon the track parameters, level of maintenance and monitoring and the degree of risk the railway administration is willing to take. A conservative approach would be to fix the T_{ALL} at T_{BMIN} value. However, a better approach would be to fix T_{ALL} higher than T_{BMIN} if the railway has good track maintenance and monitoring procedures in place.

The expression given above indicates that for a safe CWR assurance program two temperatures need to be determined:

- (i) T_{ALL} which is the allowable temperature rise above the neutral temperature for a given set of track and vehicle parameters. The single most significant factor which will govern T_{ALL} for a given set of track and rolling stock parameters is the lateral ballast resistance. The relationship between T_{ALL} and the lateral ballast resistance will be in the form of a graph. This could be given to the field maintenance engineer to enable him to predict the allowable temperature rise over the neutral temperature for a given value of the lateral ballast resistance. (Fig 7.7)
- (ii) The neutral temperature or the stress free temperature of the track.

7.7 Field Determination of the lateral ballast resistance:

A convenient method to determine the lateral ballast resistance per sleeper has been developed in the USA. It is called the single tie push test (STPT).

Test Methodology: The rail is freed from the sleeper by removing the fastenings and using a hydraulic jacking equipment the tie is pushed transversely to the track. With load transducers and gauges, the loads and corresponding displacements are recorded. The plot gives the maximum lateral resistance of ballast. For getting the average value the test could be performed on 3 sleepers over a 50 feet length. Once the lateral ballast resistance value is obtained

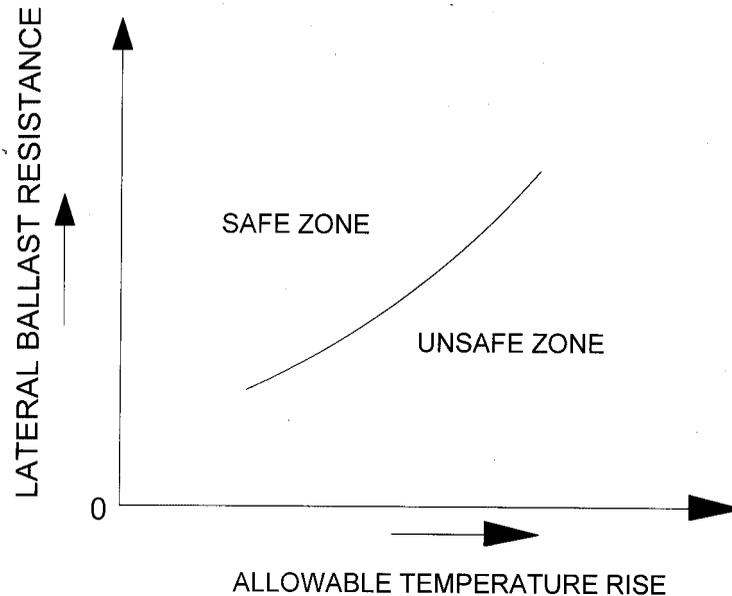


Fig. 7.7 Relationship between T_{ALL} and lateral ballast resistance.

graph could be given to the field maintenance engineer to enable him to predict the allowable temperature rise T_{ALL} over the neutral temperature.

7.8 Neutral Temperature, Its Variation and Determination

7.8.1 Introduction :

The neutral temperature t_n or the stress-free temperature t_0 is the rail temperature at which the rail is free of longitudinal stress, or longitudinal stress is zero. Till now the assumption was that once an LWR was destressed at temperature t_d , t_d was the stress free temperature of the rail. However, there is experimental evidence to indicate that the above assumption is not correct and the stress-free temperature of the rail tends to shift away from the destressing temperature. Accurate determination of the rail stress free temperature is of vital importance, because it is this temperature which determines the force level in the LWR

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$$P = AE\alpha[t_p - t_n]$$

The above expression also indicates that if due to any reason the value of t_n were to fall, it would automatically increase the compressive force in the rail and beyond a certain level could cause the track to buckle. Another way of putting it is that a change in the rail neutral temperature is equivalent to changing the force level in the rails for the same values of t_p .

7.8.2 Factors which could cause a shift in the Rail Neutral Temperature:

1) Movement of the rail in the longitudinal, lateral and vertical directions : If the CWR were to be fully constrained, then there would be no change in the neutral temperature. Since the rails cannot be fully constrained in all directions, elongation or contraction can occur whenever the track is subject to train and environmentally induced loads. Railway track motions relevant to t_n variation occur in the following three basic kinematic modes :

- (i) Rail longitudinal movement
- (ii) Track lateral shift
- (iii) Track vertical settlement.

Consider a CWR being laid at Temperature t_L and there is no rail longitudinal force at this temperature. Assume that the rail displacements (u, v and w in the longitudinal, lateral and vertical directions respectively) are measured with respect to an initial equilibrium configuration when the rail temperature is t_L . These displacements may be due to a number of causes, and in many cases are not recoverable due to the inelastic nature of the ballast. From the displacements, the longitudinal strain ϵ_x in the rail at any given temperature t_p can be calculated from the fundamental equations of theoretical mechanics.

$$\epsilon_x = - \left[\frac{\partial u}{\partial x} + \frac{1}{2} \left(\frac{\partial v}{\partial x} \right)^2 + \frac{1}{2} \left(\frac{\partial w}{\partial x} \right)^2 \right] + \alpha [t_p - t_L]$$

$\alpha [t_p - t_L]$ is a compressive strain taken as positive in the longitudinal 'x' direction.

The force in the CWR at t_p in the longitudinal 'x' direction will be

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$$P = AE \epsilon_x$$

$$= AE\alpha \left[t_p - \left\langle t_L + \frac{1}{\alpha} \left\{ \frac{\partial u}{\partial x} + \frac{1}{2} \left(\frac{\partial v}{\partial x} \right)^2 + \frac{1}{2} \left(\frac{\partial w}{\partial x} \right)^2 \right\} \right\rangle \right] \text{-----1}$$

If t_n is the neutral temperature of the CWR then

$$P = AE\alpha [t_p - t_n] \text{-----2}$$

Comparing equations 1 and 2 it is quite evident that

$$t_n = t_L + \frac{1}{\alpha} \left\{ \left(\frac{\partial u}{\partial x} \right) + \frac{1}{2} \left(\frac{\partial v}{\partial x} \right)^2 + \frac{1}{2} \left(\frac{\partial w}{\partial x} \right)^2 \right\}$$

where $\frac{\partial u}{\partial x}$, $\frac{\partial v}{\partial x}$, and $\frac{\partial w}{\partial x}$ are all tensile strains.

If the movement of the rail leads to additional tensile stress, then the neutral temperature will increase beyond t_L . On the other hand, if u , v and w the rail displacements cause compressive strains, they will cause t_n to drop below t_L .

On a curve of radius 'R' if the track is shifted by an amount equal to 'V'

$$\text{then } t_n = t_L + \frac{1}{\alpha} \left\{ \left(\frac{\partial u}{\partial x} \right) + \frac{V}{R} + \frac{1}{2} \left(\frac{\partial v}{\partial x} \right)^2 + \frac{1}{2} \left(\frac{\partial w}{\partial x} \right)^2 \right\}$$

Movement of the rail in the x, y and z directions may be due to the following reasons :

- i) Rail Longitudinal Movement – This may occur due to train action (braking and acceleration) or due to wheel rolling action.
- ii) Track Lateral Shift – This may occur due to bogie hunting action, by wheels negotiating a lateral imperfection or in case of curves, by vehicles operating

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in excess of or below the balance speed. For tangent track, the effect of lateral movement on the neutral temperature shift is likely to be small. For curved track, however, the shift can be significant.

- iii) Track vertical Settlement – Vertical wheel loads can induce differential settlement of the ballast which would cause development of longitudinal strains in CWR specially for new or recently surfaced tracks.

Apart from these displacements which could cause a shift in the rail neutral temperature, two more factors could cause a shift. These are :

1. Rail/Track Maintenance – Track maintenance activities involving lining, lifting, removal or application of rail anchors, trackline repairs all will cause a shift in neutral temperature.
2. Rail “Rolling Out” – Due to vehicle loads, plastic deformation occurs in the top layers of the rail head. Experiments conducted by the British Rail showed rolling out of rail is pronounced in first 3 months after laying new rails and continues for about an year. The basic mechanism involved is that the rolling contact loads change the residual tensile stresses in the top layer of new rails into compressive stresses. The rail residual stresses will affect the neutral temperature of the rail (Fig 7.8).

7.8.3 Neutral Temperature Measurement

7.8.3.1 A satisfactory neutral temperature measuring device should satisfy the following fundamental requirements :

1. The measuring instrument should be portable and not permanently attached to the rails.
2. The instrument should give absolute values and not relative values. Site specific calibration should not be involved.

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3. The technique should be independent of longitudinal residual stresses in rail. The residual stresses are not associated with the rail longitudinal force since they are self-equilibrating in the sense that their resultant force and moment are zero. As a result, any technique which relies on measurement of local stresses for the longitudinal force can have large errors.
4. The technique should be non-destructive.
5. The technique should be fairly accurate with measurements within $\pm 1^\circ\text{C}$.

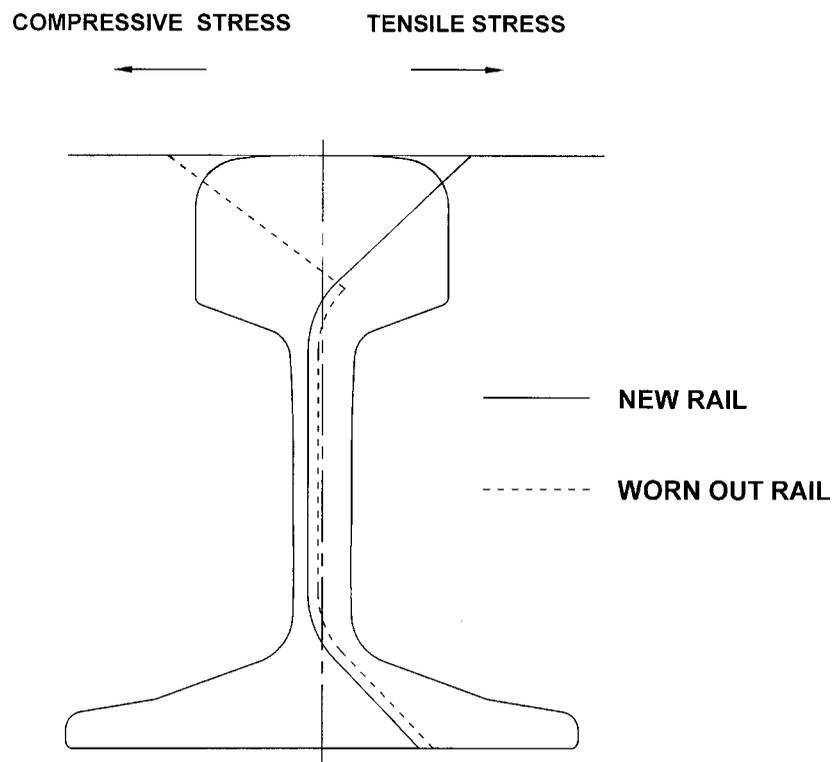


Fig. 7.8 Rail residual stresses

(123)

7.8.3.2 Of the number of techniques available for neutral temperature measurement, the following can be considered as reasonably developed :

- Berry Gauge – Simple mechanical gauge to measure change in length.
- British Rail Vibrating Wire – Measures the rail force as a function of the frequency of a wire vibrating in a hole in the rail web.
- Strain Gauge – Uses a four arm Wheatstone bridge to measure the rail strains.

Techniques under Research –

1. Flexural wave propagation
2. X-ray defraction
3. Accousto-elastic
4. Magnetic coercion
5. Barkhausen Noise – This principle is being used in Rail Scan Equipment.
6. Electromagnetic Accoustic Transducers
7. Laser 'Spackle'

7.8.3.3 Rail Uplift Method :

A new approach based on rail beam column response has shown considerable promise. It is based upon the fact that if the rail can be held at two points at some distance apart and a concentrated load applied at the centre of this portion, the rail behaves like a beam column and its deflection is influenced measurably by the longitudinal load in the rail. Clearly a compressive longitudinal load will increase its deflection, whereas a tensile load will reduce it.

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Besides the longitudinal force, the deflection is dependent on the rail flexural rigidity, EI, applied load Q and the nature of the end constraints. It is necessary to design a rig such that for all locations and measurements, the end conditions are sufficiently repeatable. As far as the end conditions are concerned, they depend upon the nature of constraint provided by the rig. Generally, the conditions are elastic supports (in between pure simple supports and completely fixed supports). Fixed support conditions improve the sensitivity, but need large applied loads. Repeatability of the end conditions is an important consideration for successful application of the technique.

The deflection Δ is given by

$$\Delta = \lambda \frac{QL^3}{EI} \frac{1}{1 - \frac{C}{P_c}} \text{-----(1)}$$

where C is the longitudinal compressive force in the rail

Q = Vehicle load applied at centre of rail.

λ = Numerical constant value depending on the end conditions,
 P_c = Critical buckling load for the beam column of length 2L for the specific end conditions.

The first factor in the above equation represents the deflection under the concentrated load in the absence of any longitudinal rail force. The second factor is the magnification factor due to longitudinal force.

The above equation shows that for a given value of C (rail longitudinal force), Q and Δ are proportional to each other. This is depicted in the given figures (Fig 7.9 & Fig 7.10)

7.8.3.4 VERSE METHOD

Practical use of this principle has been made in the technique called 'VERSE' developed by VORTOK International, UK and AEA Technology Rail. The equipment comprises of a frame featuring a hydraulic lifting device, a load transducer and a displacement

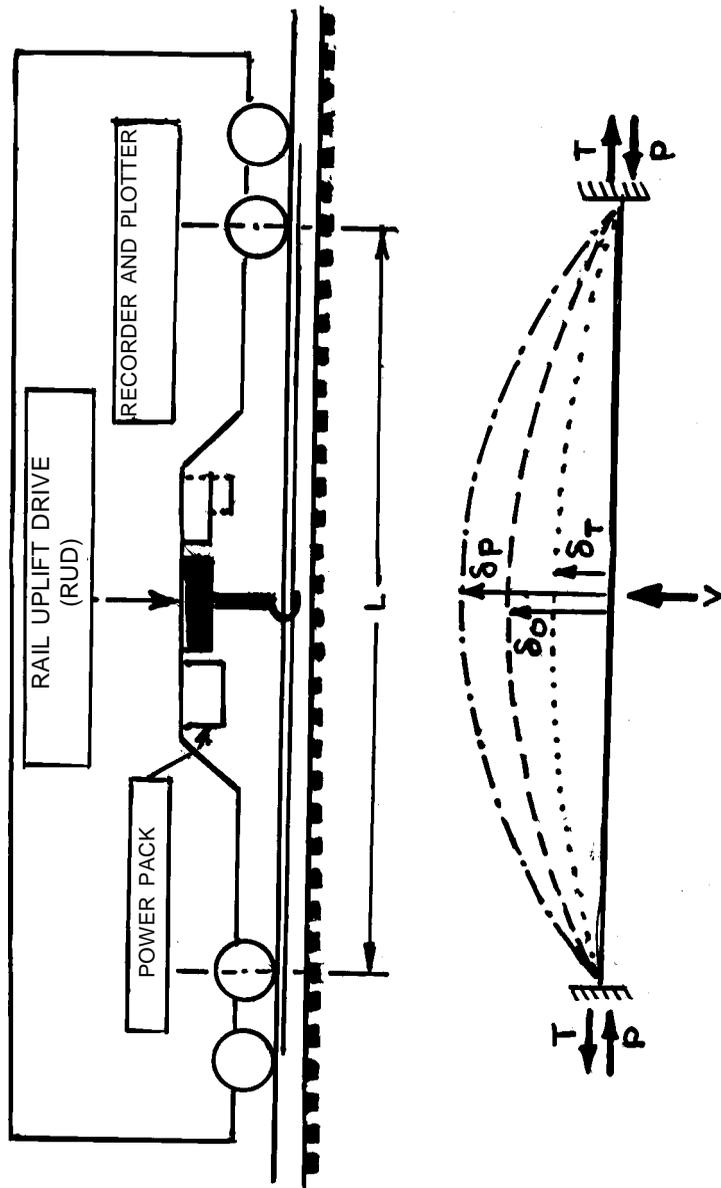


Fig. 7.9 Rail Uplift Method.

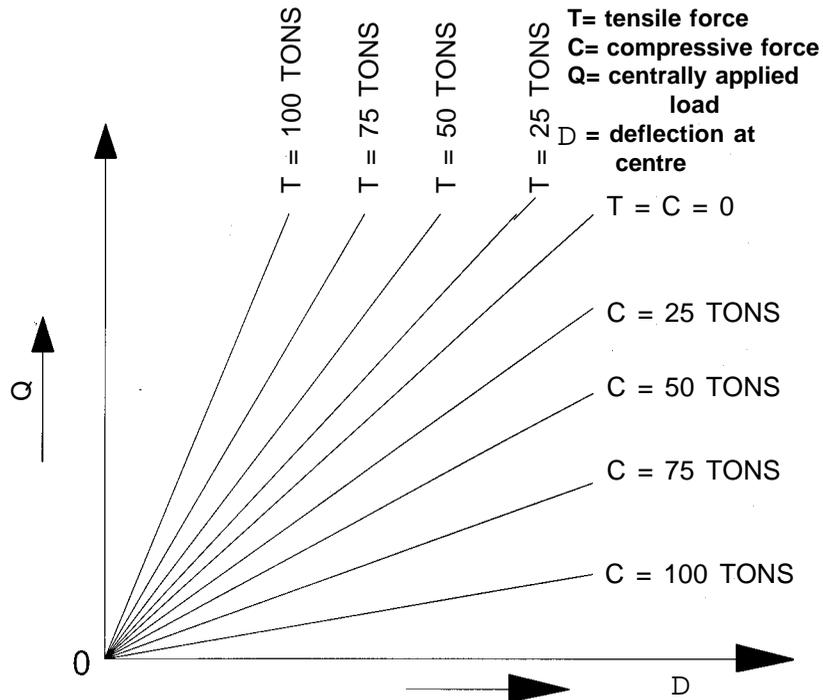


Fig. 7.10 Graphical Method for neutral temperature determination.

transducer. The measurement systems are connected to a rugged handheld computer.

The rail must be in tension at the time of measuring the stress free temperature (SFT). Taking measurements requires around 30 m of rail to be unclipped and placing rail support spacers at 10 m on either side of the measuring point (Fig 7.11). A maximum force of one tonne is applied and the load and displacements measured by the transducers relayed to the handheld computer. The measured data along with some other data such as ambient rail temperature, rail profile and height of rail is fed into the computer to obtain the SFT result. The height of the rail is included to take account of the

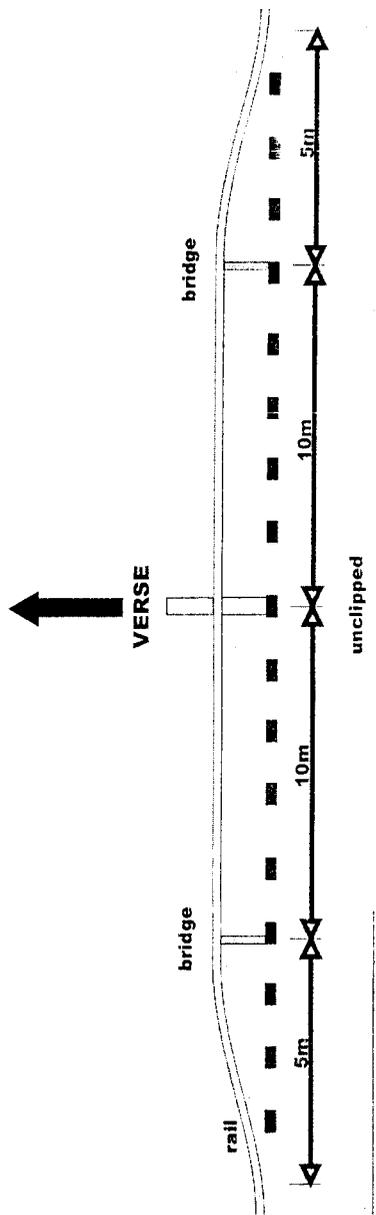


Fig. 7.11 VERSE METHOD FOR NEUTRAL TEMPERATURE DETERMINATION.

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rail head wear and rail grinding which will naturally affect the stiffness of the rail. Validation of VERSE technique has been carried out by AEA Technology, one of Britain's leading technology companies.

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