

Thirty Years of Experience with Continuous Welded Rail on the French National Railroads

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For many decades, railroad technology was used to set up tracks with jointed rails and lengths in accordance with rolling technology and handling possibilities. With increasing loads and speeds and improvements in rolling, welding, and fastening technology, railroad engineers became interested in eliminating joints, which have drawbacks in the track and in controlling rising maintenance costs. A French railroad engineer in the early 1930s carried out his first studies and reflections. His conclusions and tests after World War II led step by step to 100 mi of welded track in France. The results of using welded rail are cost-effectiveness and greater comfort.

During the early development of railroad technology, rail lengths were limited by the rail manufacturing process, first to 1.5 m (5 ft), then to 8 m (26 ft), to 12 m (40 ft), to 18 m (60 ft), and finally to 36 m (118 ft). Today's rolling plants produce lengths of 36 m (120 ft) or even 72 m (240 ft).

After the end of World War II in 1945, new developments in rail welding on site and in plants and improvements in fastenings allowed continuous welded rail (CWR) technology to be developed and to become the worldwide standard for laying track. CWR is in constant development on the French railroad network, with an ever-increasing total mileage (Figure 1). These improvements are the results of a theoretical approach in 1932 and 1933, which was confirmed by theoretical and experimental research after World War II and developed as shown in Figure 1.

JOINTED TRACK BEHAVIOR

Rails are interrupted at regular intervals to allow expansion gaps to cope with changes in temperature. Variation in rail length is not due to a simple expansion of the material but to a mixing of expansion and stresses from the friction of the track on the ballast bed or the friction between the rail and the tie. Usually it is a composition of the two levels of friction according to the fastening quality and the ballast quantity. The compression force in a rail length is equal to

$$F = f_o + \frac{L}{2} \times r$$

where

- F = compression force,
- f_o = friction between rail and joint bars (joint assembly),
- L = length of the rail, and
- r = longitudinal friction coefficient of the track in the ballast.

Compression forces in a rail are shown in Figure 2.

Friction is not a linear function, and therefore length variations are phased out with variations of temperature. If the gap between two rails is suppressed, the rails become effectively continuous and the compression force increases according to the rising temperature.

$$F' = ES \alpha \Delta t$$

where

- S = rail cross sectional arc
- E = Young's modulus,
- Δt = increasing temperature, and
- α = steel expansion coefficient.

If this situation occurs, the compression force at the joint could be high, and the track stability has to compensate to avoid buckling. This track stability is mainly solicited in curves or misalignments where a transverse component of the compression could be a major factor. Moreover, in a jointed track the total inertia of the rail is cut and replaced by two joint bars, which are not a sufficient substitute, resulting in a weak joint. The track stability is always unfavorable in a jointed track, and it is important to keep sufficient gaps between rails to avoid stresses in the rails during hot weather, which involves inspection and maintenance.

From the vertical standpoint, a weakness in track stability can be more damaging to the surface of the track than a potential buckling for the same reason—lack of inertia.

Maintenance of maximum rail inertia is necessary to (a) reduce the maintenance costs; (b) improve passenger comfort; and (c) save the track components (rails, ballast, fastenings), maintaining a good leveling.

CWR DEFINITIONS

A CWR is an unlimited length of rail, interrupted only for technical, on-site reasons such as long span bridges. The rail

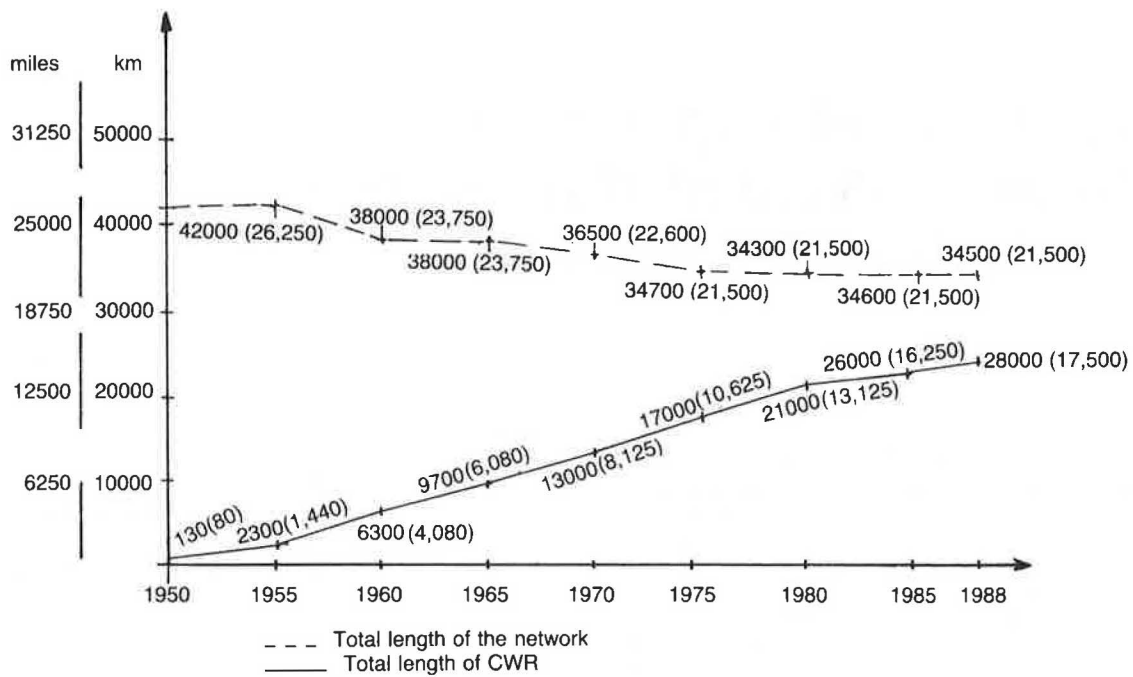


FIGURE 1 CWR on the French National Railroads.

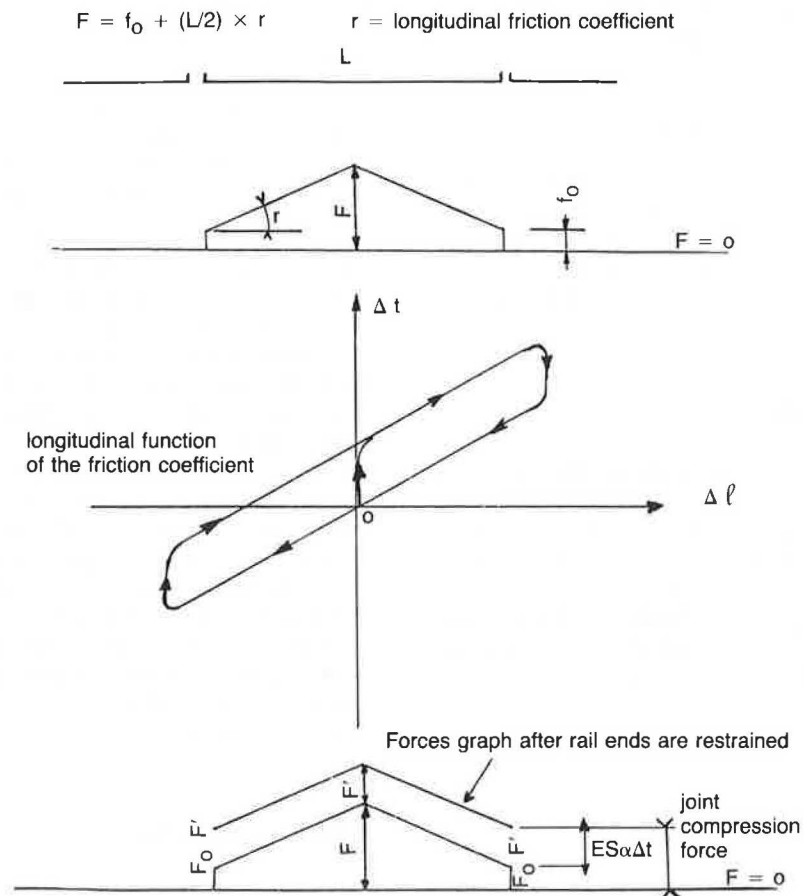


FIGURE 2 Compression forces in rail.

is secured on the ties by resilient fastenings, in order to keep the rail clamped to the tie under any circumstances. No movement between rail and tie is allowed; all friction and longitudinal displacements must be between the tie and the ballast. The longitudinal restraining force between the rail and tie must be more efficient than the longitudinal restraining force of the track in the ballast bed.

The track assembly (tie, rail, pads, fastenings) is laid on hard crushed stone—the ballast—with optimized quantities and specified profiles. In the major part of a length of CWR, variations of the temperature induce only stress variations (without movements), but in two areas of the CWR both stresses and length variations occur. These stresses (forces) and length variations must be contained by the track, and the quality of the track that copes with the necessity to avoid buckling is its stability, which primarily takes into account the transverse direction. The two areas of a CWR affected by longitudinal displacements according to variations in temperature are the ends.

The friction coefficient between the track (ties) and ballast restrains compression forces gradually, the total forces being blocked on lengths that are functions of the rail temperature changes (Δt) between the laying temperature or stress-free temperature (t_o) and the actual temperature of the rail (t_r).

CWR definitions are shown in Figure 3. In Figure 3, the compression force (F) in breathing zones 1 and 3 is

$$F = ES \alpha \Delta t - \int \frac{du}{dx} ES$$

The compression force in zone 2, the middle part blocked by the two breathing lengths, is

$$F = ES \alpha \Delta t$$

Under a simultaneous action of Δt and F , length/variation of the track element is

$$du = \left(\alpha \Delta t - \frac{F}{ES} \right) dx, \left(\frac{dl}{L} = \frac{\sigma}{E} = \frac{F}{SE} \right)$$

$$\frac{du}{dx} = \alpha \Delta t - \frac{F}{ES}$$

from which

$$F = ES \alpha \Delta t - \int ES \frac{du}{dx}$$

The F graph is presented with a simplified function of the friction coefficient, which is normally a function of the displacement

$$\frac{d\phi}{dx} = f(u)$$

where

ϕ = friction coefficient,
 u = track base displacements, and
 x = length of rail.

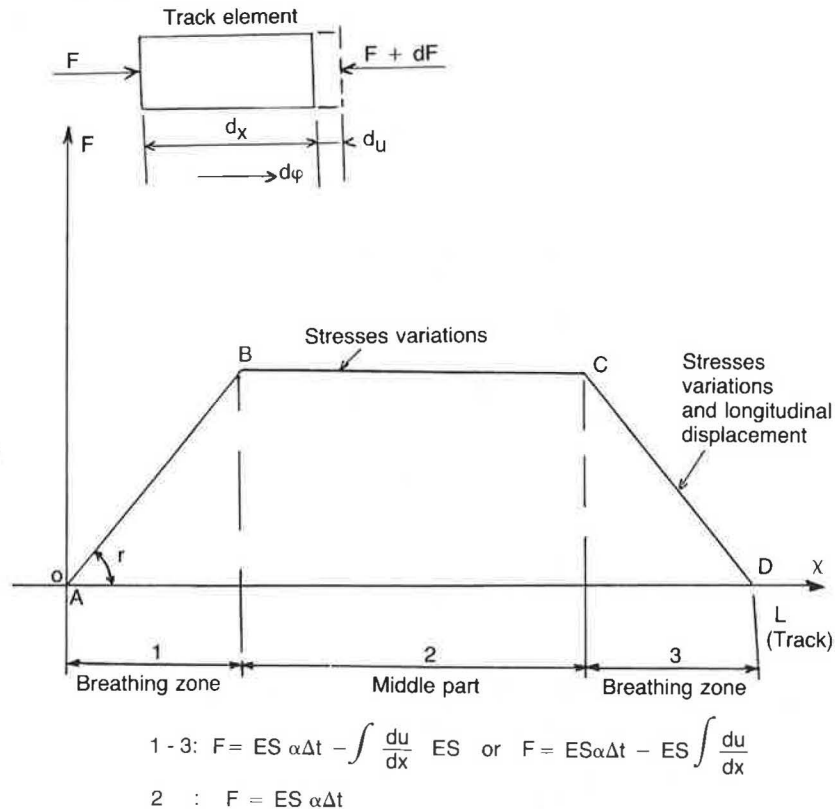


FIGURE 3 CWR definitions.

The friction coefficient is shown in Figure 4. The simplified function $d\phi/dx = f(u) = \pm r$ is set up to simplify assumptions in the theoretical approach. A normal function is more complicated, but this approach is sufficient for a correct evaluation of the longitudinal phenomena.

If we imagine a rail that is laid stress free with sufficient accuracy, the longitudinal force distribution is presented in Figure 3. Both ends of the CWR are in movement, and points *A* and *D* are free of stress. Breathing lengths *A-B* and *C-D* experience stress (force) variations and movement. Between points *B* and *C*, only stress (force) variations occur.

The behavior of the CWR according to temperature variations is shown in Figure 5. Increasing temperature from the stress-free temperature to a higher temperature t_1 (t_0 is the stress-free temperature),

$$F_1 = ES \alpha (t_1 - t_0),$$

with a linear connection to $F_1 = 0$ and with the simplified friction coefficient between ballast and ties $tg \alpha = r$.

If temperature t_2 occurs, the horizontal part of the F line will decrease to

$$F_2 = ES \cdot \alpha (t_2 - t_0)$$

or

$$F_1 - F_2 = ES \alpha (t_1 - t_2)$$

or

$$\Delta F = F_1 - F_2 = ES \alpha (t_1 - t_2)$$

In order to understand what happens in the breathing length it is assumed that the end of the CWR is restrained (i.e., no movement is possible). The graph will be parallel to the first one (t_1), with a gap.

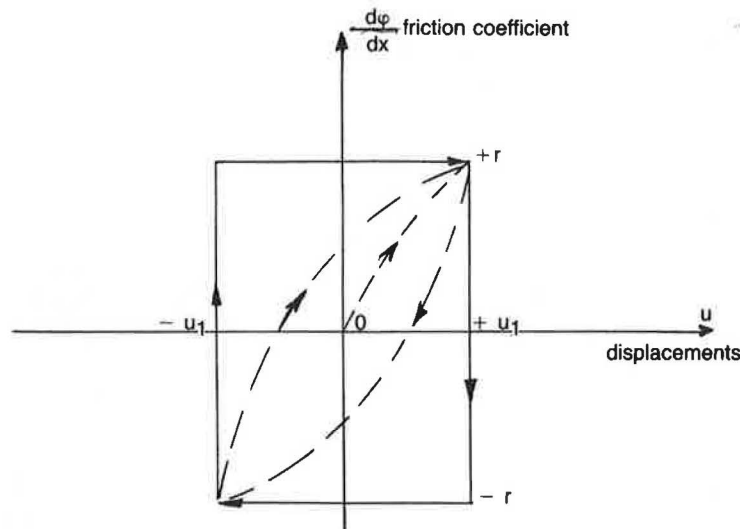
$$\Delta F = ES \alpha (t_1 - t_2)$$

If the restrained end is again considered free (i.e., there are no stresses at the end), all the points of the CWR between *A* and *C* will have tensile stresses, and the end will have movement between *A* and *D* toward the middle part. The graph shows irregularities in the breathing length after only one cycle in temperature. A total scope of the expansion zone behavior would show complete movement of the expansion zone end between two points according to the maximum temperature changes of the year. Between these two extreme positions only slight variations closely linked to the general sketch occur.

CWR BEHAVIOR AND TRACK STABILITY

In the middle part of the CWR, variations of temperature involve only stress variations, which give the rail a general state of balance comparable to a long beam under compression stresses in potential buckling situations.

A track is different, however, because of rail fastening assemblies and specified ballast layer profiles. This special rail-fastener-cross-tie frame must withstand temperature stresses without noticeable geometric defects, which could affect traffic. The stability of the track allows it to keep its geometry during all the temperature variations throughout the year. Different factors confer stability to the track, including



$$\frac{d\phi}{dx} = - \frac{dF}{dx} \text{ is the real friction coefficient in relation with the displacement } u$$

$$\frac{d\phi}{dx} = \Delta f(u)$$

To simplify assumptions we use this simplified function $f(u) = \pm r$

FIGURE 4 Friction coefficient.

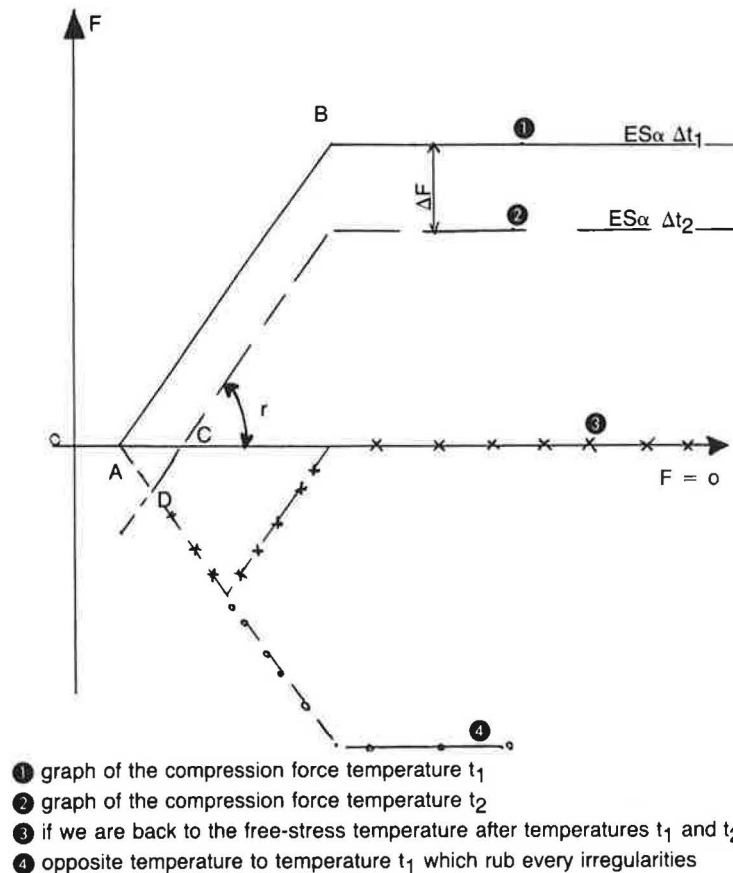


FIGURE 5 CWR behavior as affected by temperature variations.

- Rail-fastening assemblies;
- Ballast profiles, with designed depth, superelevated (heaped) shoulders, and ballast particle size (gradation);
- Track weight and rail inertia; and
- Track geometry quality (alignment imperfections).

An efficient rail-fastening assembly is necessary to keep the rail securely fastened on the tie to avoid any longitudinal movement between the rail and tie and to restrain the rotation of the rail base on the tie (torsional resistance). This value is defined by a torque per meter of track (statistical mean of tests), in accordance with the angle of rotation. Movement of CWR ends is shown in Figure 6. The limits of the expansion zone are demarked by $+u_1$ and $-u_1$. C , the torque, is

$$C_{\text{torque}} = K\alpha$$

where

- C = torque density per meter of track (kN),
- α = rotation angle (rad), and
- K = coefficient (kN/rad).

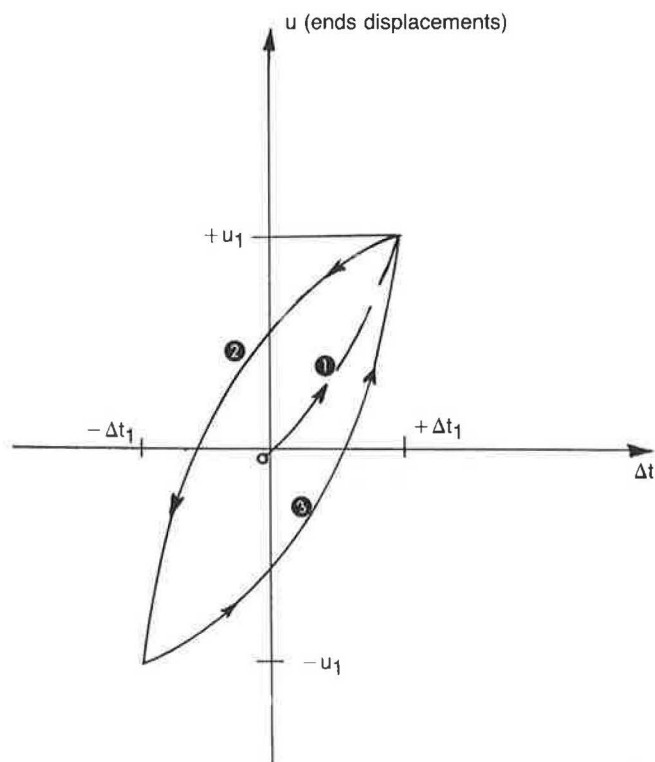
Taken into account is the linear part of the function $C = f(\alpha)$, before the horizontal part of the experimental C value. Torsional resistance of the fastenings is shown in Figure 7.

Specified ballast profiles give the track a sufficient transverse (and longitudinal, as shown before) resistance to avoid

geometric defects and buckling. From the longitudinal standpoint, the ballast section must accommodate the longitudinal rail forces operative in the breathing lengths. The vertical resistance must be examined from the track modulus standpoint, not in terms of the friction coefficient. The most important part of the resistance is the lateral resistance, which withstands the lateral displacement of the track caused by transverse forces. The transverse resistance, sometimes characterized as τ_r , is represented in Figure 8 by the values of kN/m. When considering CWR track, two factors must be taken into account—nonconsolidated track and consolidated track. A value for the second factor is obtained after a minimum of 100 000 tonnes of traffic or dynamic stabilization. Values for the transverse resistance of two segments of rail are presented in Table 1. Note the difference between concrete and timber in weight and the difference between consolidated and non-consolidated track, particularly in the timbered track.

The two curves in Figure 8 are different in value because the horizontal part of the simplified one must be considered as a maximum and in terms of rigidity for the first part of the graph. The real function, bounded to the friction coefficient of the track frame on the ballast, is simplified to allow a reasonable and theoretical approach. The track weight is essential for the value of the transverse resistance of the track.

Finally, control of track geometry imperfections is essential for acceptable track behavior: the lower the quality of the geometry, the more the track is prone to buckling. The am-



$+u_1$ and $-u_1$ provide limits of the expansion zone

FIGURE 6 Movement of CWR ends.

plitude of defects in track geometry, which increases with compression forces (the wavelength of the defect being unchanged), leads to the point of buckling. The relationship between lateral geometric defects and longitudinal compression forces is illustrated in Figure 9. As the defects increase, the lateral track resistance becomes more involved and reaches its limit at the point of buckling. At that moment the graph shows the horizontal zone of the lateral resistance of the track. A critical amplitude of the defect can be defined to cope with the maximum temperature variation between the stress-free temperature and the maximum temperature the track can reach before buckling.

THEORETICAL APPROACH TO TRACK STABILITY

Vertical Stability

It is assumed that the track is perfectly level when the stress-freeing operation is carried out. The occurrence of leveling defects takes place after a certain amount of traffic, in the form of subgrade or soil deformation.

A longitudinal defect is shown in Figure 10. For a compression force F the ballast reaction r becomes less than the normal weight of the track. This F value is calculated with the equilibrium equation of a track element. For each F value of longitudinal force, the maximum weight relief is determined by a critical wavelength of the defect, which tends to maxi-

mize its amplitude. When all calculations are made, this wavelength is

$$L = 8.88 \left(\frac{EJ}{F} \right)^{1/2}$$

where

J = rail vertical inertia,
 E = Young's modulus, and
 F = longitudinal compression force.

The ballast reaction for a meter is

$$r = \bar{w} - b \frac{F^2}{4EJ}$$

where \bar{w} is the weight of the track (in kilonewtons per meter). Weight increases at the bottom of the wave and decreases at the top. Loss of track weight can be taken into account with this result to reduce the lateral resistance of the track. For the French National Railroads, assuming loss of track weight, the calculation for the maximum defect is

$$2b = \frac{L}{1,000}$$

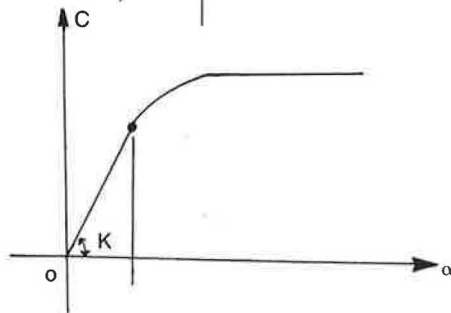
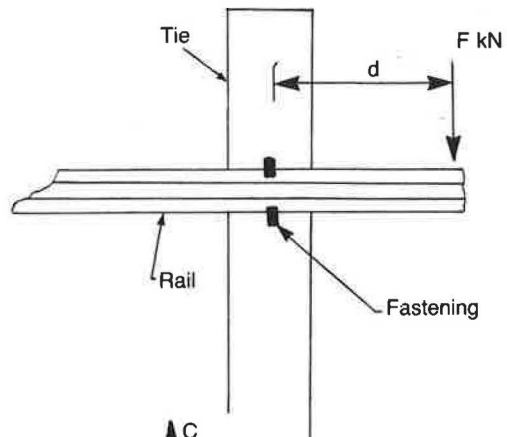
$$b = \frac{L}{2,000}$$

Transverse Stability

The essential factors that directly affect the transverse stability of the track are

- Rail inertia,
- Torsional resistance of the fastenings,
- Ballast profile resistance to the transverse forces in the linear part of its stiffness, and
- Track geometry quality.

Rail inertia is well known and need not be discussed here. The torsional resistance $C = K\alpha$, proportional to the rotation of the rail on the tie, is an experimental value. The graph of the transverse resistance of the ballast is taken in the part of linear displacements (function of the transverse force). It is also an experimental value, with two figures, in the case of consolidated track and nonconsolidated track. For example, if a track has no alignment defect and is perfectly straight, it is impossible to trigger buckling with normal climate conditions. The longitudinal forces ($F = ES\alpha \cdot \Delta t$) would be too moderate as far as the temperature variations are concerned between the stress-free temperature and the maximum rail temperature. An alignment defect at the stressing (or laying) operation often results in track buckling. The amplitude of the defect at the stress-free temperature increases with the temperature, and the wavelength remains unchanged. This increasing amplitude gradually absorbs the transverse resistance of the ballast, up to the buckling point. To carry out a



$$C = K\alpha$$

Units:
for C , $\frac{\text{m} \times \text{kN}}{\text{m}} = \text{kN}$

for K , $\frac{\text{kN}}{\text{rad}}$

Examples of current figures: - fastening on concrete ties: $K = 100 \text{ kN}$
- fastenings on timber ties: K is situated between 45 kN and 150 kN

(These values for K are typical for SNCF fasteners.)

FIGURE 7 Torsional resistance.

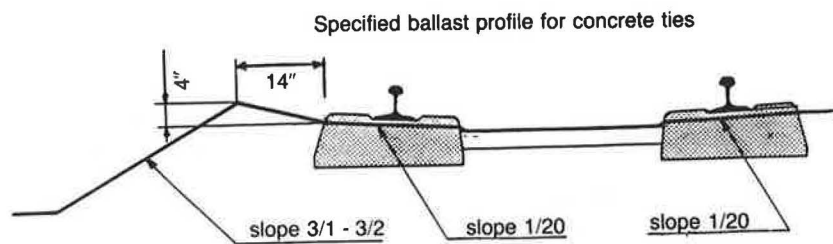
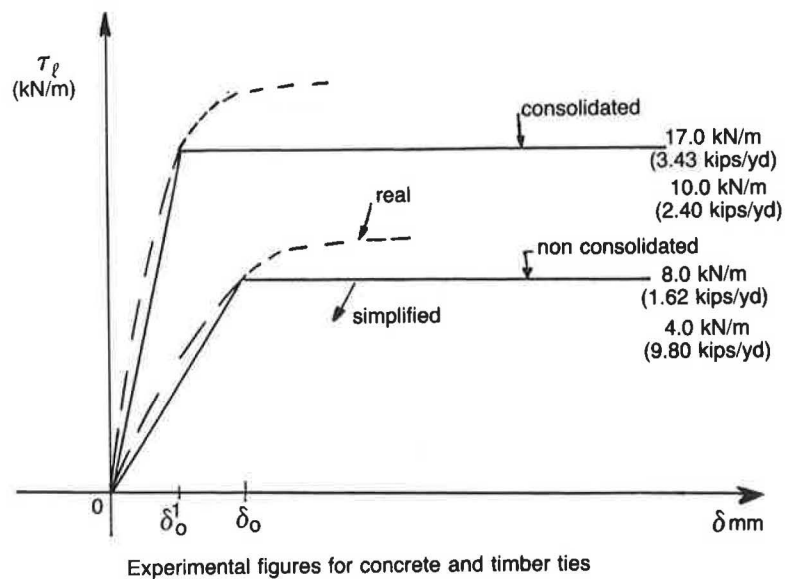


FIGURE 8 Transverse resistance of the track.

TABLE 1 TRANSVERSE RESISTANCE VALUES OF TWO SEGMENTS OF RAIL

TIE	RAIL	TRACK			
		NON CONSOLIDATED (kN/m) kips/yard		CONSOLIDATED	
		BALLAST STANDARD	PROFILE REINFORCED	BALLAST STANDARD	PROFILE REINFORCED
TIMBER	110 lbs/y	(1,700) 0,34	(1,940) 0,39	(5,200) 1,03	(5,750) 1,14
CONCRETE	"	(3,500) 0,70	(3,920) 0,78	(5,900) 1,17	(8,500) 1,69
TIMBER	120 lbs/y	(1,770) 0,35	(2,010) 0,40	(5,200) 1,03	(5,750) 1,14
CONCRETE	"	(3,500) 0,70	(3,920) 0,78	(5,900) 1,17	(8,500) 1,69

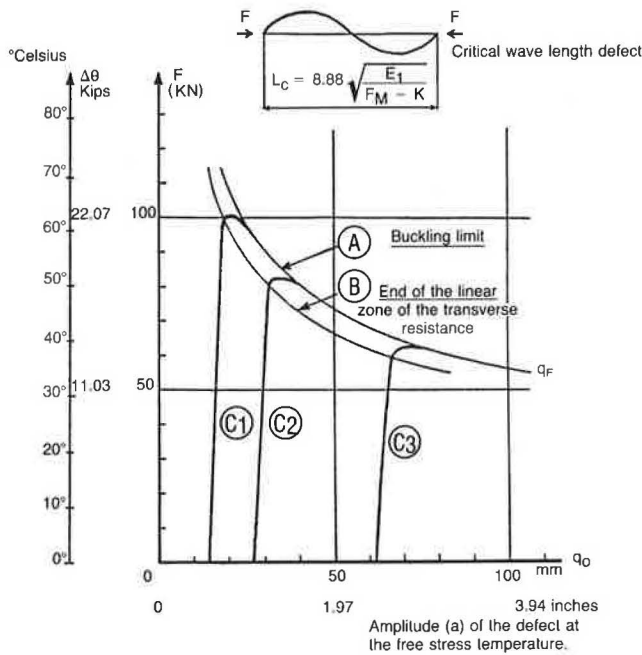


FIGURE 9 Relationship between lateral geometrical defects and longitudinal compression forces.

theoretical approach to the problem, a sinusoidal shape of the defect in a tangent track is considered.

The defect is defined by

$$Y = a \cos \omega x$$

$$\omega = \frac{2\pi}{L}$$

where L is the wavelength of the defect. It is possible to define the amplitude of the defect with a versine value, for example, a 10 m chord:

$$f = a (1 - \cos 5\omega)$$

The maximum value of the amplitude (a_{\max}) of the defect at the stress-free temperature is

$$a_{\max} \leq \frac{\tau_t - \frac{F}{R}}{EI \omega_{\max}^4 \left(1 + \frac{F \omega_{\max}^2}{k' - EI \omega_{\max}^4} \right)} \quad (1)$$

This maximum function occurs with a critical wavelength L_{\max} .

$$L_{\max} = 8.88 \left(\frac{EI}{F - K} \right)^{1/2}$$

$$\omega_{\max} = \frac{2\pi}{8.88} \left(\frac{F - K}{EI} \right)^{1/2}$$

The formula in Equation 1 enables the maximum value of longitudinal forces or the maximum value of a defect to be found, according to the chosen assumptions. Previously, it was found that the track has a weight loss in the case of the track level defect. A normal value of this weight loss (i.e., loss of transverse resistance $\Delta \tau$) must be taken into account.

Finally, geometric defects of the rail, that is, manufacturing process defects of the rail at the rolling plant, and the welding process defects must be examined. They have an important influence on the real transverse resistance of the track. The lateral force used by the ballast profile to correct these defects reduces its total value. It is assumed that a rail geometric

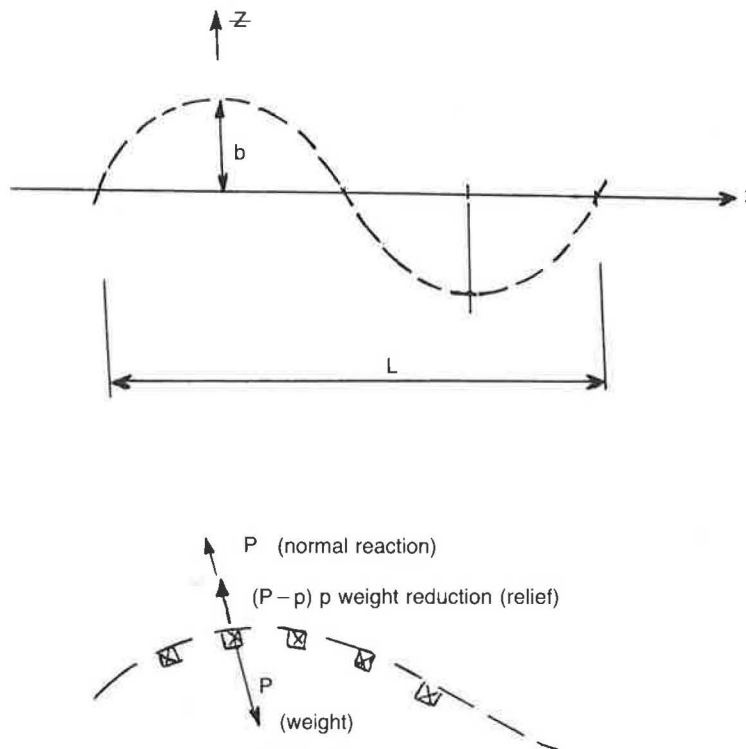


FIGURE 10 Longitudinal defect in CWR.

defect of $1/1,000$ rad or a defect of an amplitude of 2.5 mm (1/10 in.) with a 10-m wavelength is the maximum value allowed in CWR track. The transverse resistance of the ballast layer is reduced as follows to take account of geometric defects of the rails:

$$\Delta_2 \tau = a_2 \omega_2^2 (EI \omega_2^2 + K)$$

where a_2 is the amplitude of the defect and ω_2 equals $2\pi/L_2$. A 20 percent reduction of the vibrations (0.80 coefficient) must also be taken into account. The final value is

$$\tau' = 0.8 (\tau_1 - \Delta_1 \tau) - \Delta_2 \tau$$

Equation 1 when used for tangent tracks becomes

$$a_o \geq \frac{\tau'_l - \frac{F}{R}}{EI \omega_o^4 \left(1 + \frac{F \omega_o^2}{k' - EI \omega_o^4} \right)}$$

where

- $\left(\tau'_l - \frac{F}{R} \right)$ = the final transverse resistance of the track,
- τ_l = the total transverse resistance, taking into account all the reductions, and
- $\frac{F}{R}$ = the transverse component of F in a curve with Rm radius.

Each unknown quantity can be calculated by taking into account assumptions for the others. For example, a minimum R (m) can be found in assuming a maximum alignment defect, a maximum F , and a maximum rail geometry defect. An equivalent approach can be carried out with graphs of the function

$$F_{\max} \text{ (or } \Delta t) = f(Rm).$$

Track stability is shown in Figure 11. A curve can be obtained for each transverse resistance value, and with a specified radius of a curve, it is possible to learn the maximum value for F (i.e., Δt maximum). These curves are useful to study special locations in the CWR at which longitudinal forces might be different compared with a standard situation. Some cases involve force variations in the middle part of the CWR.

SITE CONDITION EFFECTS

CWR behavior was examined previously with some assets: total length of the CWR in a similar site condition—plain line, same sun exposure, no bridge, no switch, and no tunnel. Real conditions are slightly different, and the result causes disturbances in the standard behavior of the CWR. The track stability must withstand these disturbances, which are dangerous when they increase the compression stresses. For example, when a long bridge with an uninterrupted longitudinal beam is in the central part of the CWR, a maldistribution of the stresses along the CWR is noticed at the free end, that

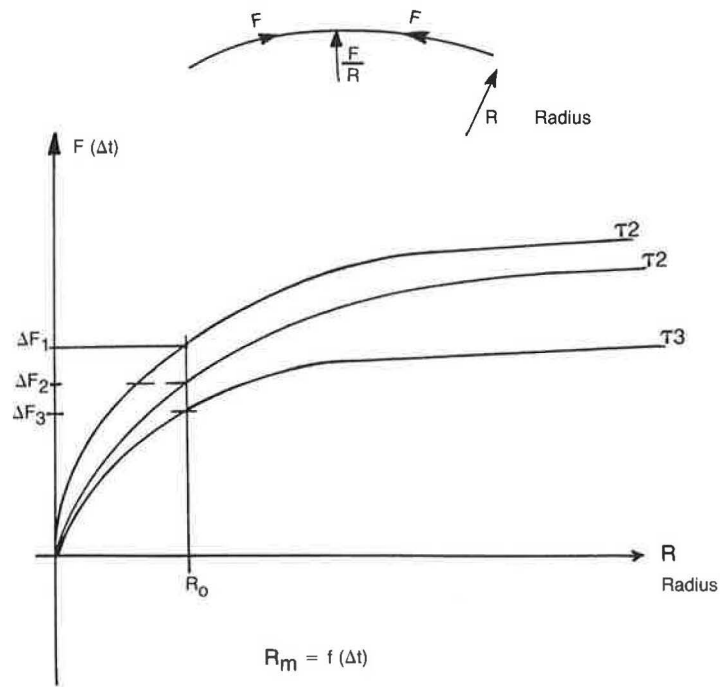


FIGURE 11 Track stability.

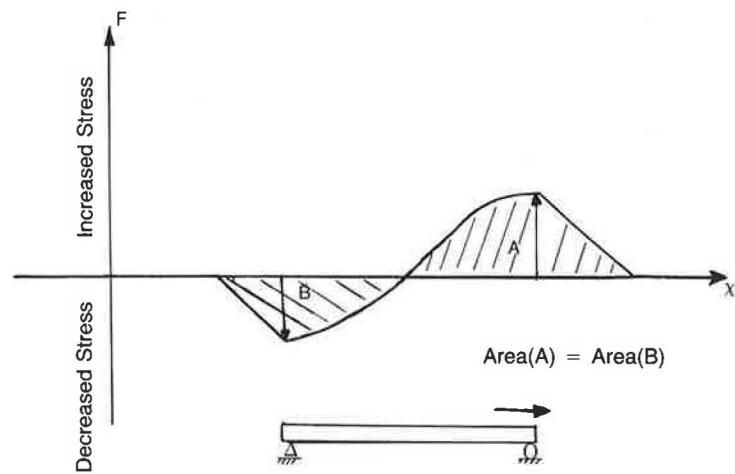


FIGURE 12 Compression stresses of CWR on a bridge.

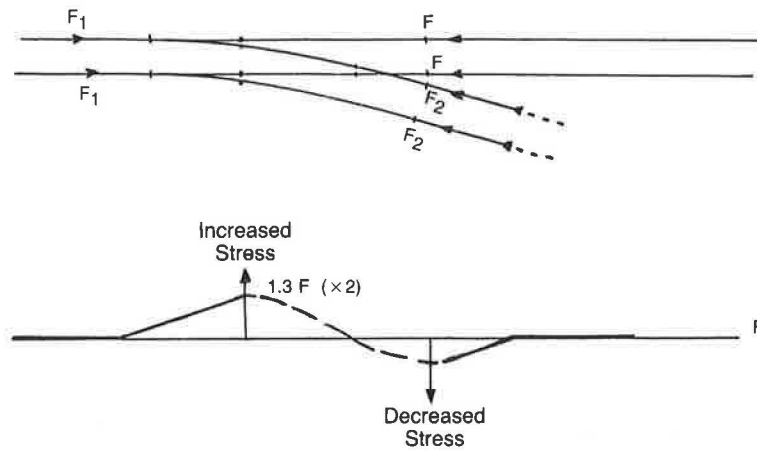


FIGURE 13 Compression stresses of CWR at a turnout.

is, stresses increase at the free end and decrease at the other. These increased stresses or (forces) are caused by the friction of the ballast on the bridge spans, which drags the track toward the free end. The distribution of stresses (F) is shown in Figure 12. This inclusion of bridges in CWR systems requires that both the radius of the track and the length of the spans be limited in order to keep the track in a state of balance. Peaks of longitudinal stresses are distributed, as they are in a breathing zone.

Other sites that affect the behavior of CWR include tunnels with welded rail throughout and turnouts with particular conditions, in which four stretches of rail become two stretches with increased compression forces in each of them. The compression stresses of a turnout in CWR are illustrated in Figure 13. The difference between the crossing heel with four rails and the toe-switch involves increased stresses at the heel of the stock-rail. The coefficient of overvalue is 1.30; the major part of the longitudinal force is blunted by flexion of the supports and friction of the rail on the bearers before the heel of the switch rail. The track stability must withstand this 30 percent increase in stresses (forces).

CONCLUSIONS

Mathematical development and all experiments done to verify the results of the theoretical approach are defined in this paper. All this work was done by French National Railroad engineers, who were primarily the first developers and who transferred the results in practical applications to the French rail network. These applications are possible with specific procedures performed in accordance with local situations and climate conditions; the first procedure applies to laying and stressing. The CWR system gives an accurate and reliable solution to problems of jointed track that relate to buckling and maintenance costs. The theoretical method to find the limit of track stability and its application, which takes into account every potential defect at the same moment (e.g., track geometry, levelings, and rail quality), gives to the track a state of balance, the high level of security coefficient it needs. Strict application of the CWR maintenance rules by all staff on the tracks, however, is the guarantee of the system's excellent reliability.