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Track – Bridge Interaction on High Speed Railways – the ONCF Experience

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Abstract: ***This paper aims to describe the Moroccan experience on Track – Bridge interaction in railway domain. In fact, that phenomenon may induce a general track instability compromising train security if it's not taken into account in the study phase.***

To that end, a numerical method, used to compute the additional longitudinal stresses on a continuously welded rail (CWR) mainly associated to relative displacements between track and deck on account of temperature variations, is presented. Therefore, the conditions of laying an expansion device will be concluded, particularly when the additional rail stresses exceed the maximum limit values defined on the current standards.

An application of that method will be performed on MHARHAR viaduct where a non-compliance of track dilatation device laying was noticed. This study will emphasize the beginning of a track grid general instability due to ballast layer deconsolidation, the all based on a track geometry measurement.

This study may be widely used in order to conceive properly a bridge supposed to receive a CWR track.

Keywords: ***Track-Bridge Interaction, Railway Bridge, Continuously Welded Rail, Rail Stress, Track Dilatation Device, Track Geometry.***

I. INTRODUCTION

Various researches have been performed in order to assess the phenomenon of longitudinal track-structure interaction in railway bridges. Jean-Pierre FORTIN [1] described a general method to determine the additional stresses on CWR laid down on an isostatic deck. He also concluded about the maximum expansive length of a deck that generates tolerable stresses on rail with as regards the limit values defined in current standards. Ramondenc, Martin & Schmitt [2] shared the SNCF experience by treating the conception of structures in order to manage this phenomenon, especially in long continuous bridges where track expansion devices are needed. Sanguino & Requejo [3] laid out different computing methods of rail-structure interaction based on the UIC Leaflet 774-3 [4] and the Eurocode 1991-2:2003 [5].

In the same vein, this paper deals with clarifying the significant impact of that phenomenon on track stability by highlighting the real behavior of a CWR observed on the occasion of a non-compliance associated to a track dilatation device laying, the all focused on two main aspects:

- Determination of the absolute deck displacements and relative displacements between track and deck;
- Determination of the additional stress on rail due to temperature variations.

Track geometry measurement will be used to spot the main areas of track instability. Consequently, a comparison between these zones and the points where the rail stress reaches its maximum values according to the theoretical study may be performed. To that end, the alignment and levelment signals of track geometry will be filtered in intermediate wavelength range [25m – 70m] as it's described later on this paper.

II. THEORETICAL STUDIES AND GENERAL DESIGN GUIDELINE

The main purpose of this section is to describe the numerical method of computing the relative displacements between rail and deck and, consequently, the stresses on the rail on an isostatic bridge in order to draw conclusions concerning the maximum dilatation length that has to be observed while conceiving a railway structure. Therefore, we may determine the eventual need of a track expansion device in long continuous bridges exceeding the said dilatation length limit and the conditions of its laying according, obviously, to the structure design. For this purpose, it's proposed to treat successively the following items:

- Behavior of CWR by determining the equations describing its behavior;
- Phenomenon of track-bridge interaction;
- Definition of the optimal structure design on long continuous bridges and the eventual need of expansive devices.

A. Continuous Welded Rail behavior

We assume an elementary length of rail on a ballasted track (Fig. 1). It's subjected to the following loads:

- Temperature variation loads;
- Ballast resistance to track displacements applied on the sleeper/ballast contact area.
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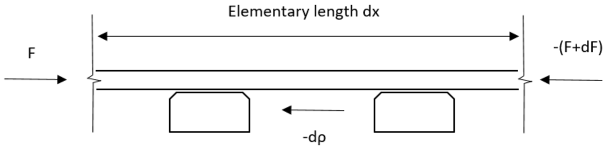

Fig. 1.Loads on an elementary length of rail on a ballasted track

The variation of the length of the said elementary rail under temperature gradient Δθ load is governed by the equation:

$$\frac{du}{dx} = \alpha.\Delta\theta - \frac{F}{E.S} \quad (1)$$

Where α is the steel coefficient dilatation, E is the steel modulus of elasticity and S is the rail surface.

Furthermore, the equilibrium of the elementary length of rail is reflected by the equation:

$$F - (F + dF) - d\rho = 0$$

That is:

$$\frac{dF}{dx} = -\frac{d\rho}{dx} \quad (2)$$

In this study, we assume that the resistance of ballast to the track displacements is linear.

The equations (1) and (2) describe the behavior of a CWR as follows:

- **Neutral zone:**

In that area, the rail displacement is almost nil, whereas the rail stress reaches its maximum value.

$$\frac{du}{dx} = 0 \quad and \quad F_M = \alpha.E.S.\Delta\theta$$

Which means:

$$\sigma_M = \frac{F_M}{S} = \alpha.E.\Delta\theta$$

- **Expansive zone:**

As it's assumed that ballast resistance to the track displacements is linear, we may write:

$$\frac{dF}{dx} = r \quad where \quad r\ is\ a\ constant.$$

Therefore, rail stress is given by:

$$\sigma = \frac{r.x}{S}$$

And the rail displacement is given by:

$$u(x) = \int_0^x \frac{1}{E.S}(E.S.\alpha.\Delta\theta - F)dx' = \alpha.\Delta\theta.x - \frac{r.x^2}{2.E.S}$$

Let's assume that L_z is the expansive zone length. The total rail displacement is given by:

$$L_z = \frac{E.S.\alpha.\Delta\theta}{r}$$

$$u(L_z) = \frac{E.S.\alpha^2.\Delta\theta^2}{2.r}$$

Numerical application:

Table- I: parameters values

α	*Δθ (°C)*	*S (mm²)*	*E (MPa)*	*r (daN/m)*
11.10^{-6}	45	7500	210000	600

$$L_z \approx 130\ m \quad and \quad u(L_z) \approx 32.16\ mm$$

Therefore, stress and displacement diagrams would be:

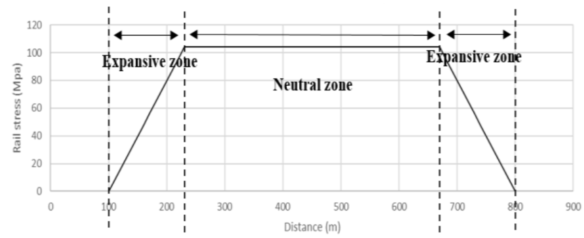

Fig. 2.Rail stress diagram on a CWR

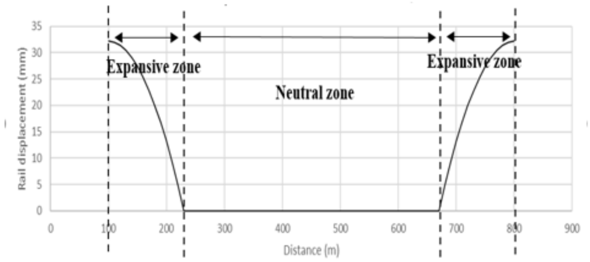

Fig. 3.Rail displacement diagram on a CWR

B. Track-Bridge Interaction Phenomenon

In general, bridges in railway domain are subjected mainly to three kind of loads that are:

- Thermal actions;
- Braking and acceleration forces;
- Vertical loads.

These loads, when applied, impose deformations in bridges that induce additional stresses in rails if they are crossed by a CWR. In this paper, the focus is given to thermal actions as long as they are closely linked to the non-compliance that will be discussed later on this paper.

Let's assume an isostatic bridge whose length is equal to L. The additional load ΔF on the rail is defined as:

$$F = \Delta F + F_M \quad where \quad F_M = E.S.\alpha.\Delta\theta$$

F_M is the load applied on a CWR in its neutral zone as it's explained earlier on this paper.

The absolute displacements of the rail and the bridge u_R and u_B respectively are given by:

$$\frac{du_R}{dx} = -\frac{\Delta F}{E.S} \quad and \quad \frac{du_B}{dx} = \alpha'.\Delta\theta' - \frac{F'}{E'.S'}$$

Where α' = bridge dilatation coefficient, S' = bridge transversal section, Δθ' = bridge temperature variation and F' = axial load on the bridge.

Therefore, the relative displacement between the rail and the bridge is given by:

$$\frac{d(u_R - u_B)}{dx} = \alpha'.\Delta\theta' - \frac{\Delta F}{E.S} \quad (S' \gg S)$$

As a result, we may derive the diagram of additional load as it's shown in figure (4) by assuming a linear ballast resistance to track longitudinal displacement.

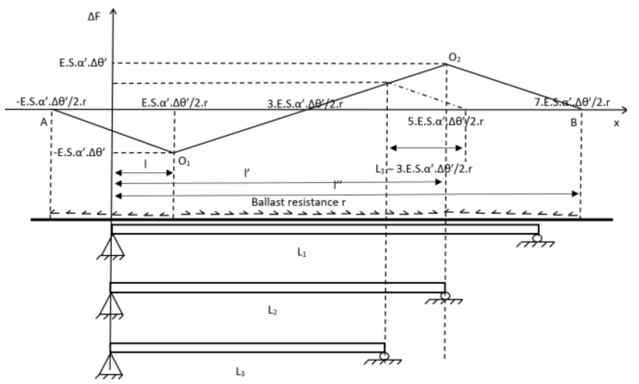

Fig. 4.Axial force distribution on rail

At the point O_1, absolute rail displacement is equal to absolute deck displacement. Consequently:

$$\frac{d(u_R - u_B)}{dx} = 0 \quad and \quad \Delta F = -E.S.\alpha'.\Delta\theta'$$

The additional load and absolute rail displacement may be written as follows:

- **Section [AO_1]:**

$$\Delta F = r.(l - x) - E.S.\alpha'.\Delta\theta'$$

$$u_R(x) = \int_{l-\frac{E.S.\alpha'.\Delta\theta'}{r}}^{x} -\frac{\Delta F}{E.S}dx' = \frac{r}{2.E.S}(x-l)^2 + \alpha'.\Delta\theta'.(x-l) + \frac{E.S.\alpha'^2.\Delta\theta'^2}{2.r}$$

As a conclusion:

$$u_R(l) = \frac{E.S.\alpha'^2.\Delta\theta'^2}{2.r} = u_B(l) = \alpha'.\Delta\theta'.l \quad means \quad l = \frac{E.S.\alpha'.\Delta\theta'}{2.r}$$

- **Section [O_1O_2]:**

$$\Delta F = r.(x - l) - E.S.\alpha'.\Delta\theta'$$

$$u_R(x) = \int_{-\frac{E.S.\alpha'.\Delta\theta'}{2.r}}^{x} -\frac{\Delta F}{E.S}dx' = -\frac{r}{2.E.S}x^2 + \frac{3}{2}.\alpha'.\Delta\theta'.x + \frac{E.S.\alpha'^2.\Delta\theta'^2}{8.r}$$

- **Section [O_2B]:**

$$\Delta F = r.(2.l' - x) - \frac{3}{2}.E.S.\alpha'.\Delta\theta'$$

$$u_R(x) = \int_{-\frac{E.S.\alpha'.\Delta\theta'}{2.r}}^{x} -\frac{\Delta F}{E.S}dx' = \frac{r}{2.E.S}x^2 - \frac{r}{E.S}(2.l' - 3.l).x + \frac{r}{E.S}.l'^2 - \frac{1}{4}.\alpha'.\Delta\theta'.l$$

l' and l'' obey to the conditions:

$$\Delta F(l'') = 0 \quad and \quad u_R(l'') = 0$$

Consequently, we conclude:

$$l' = 5.l = \frac{5.E.S.\alpha'.\Delta\theta'}{2.r} \quad and \quad l'' = 7.l = \frac{7.E.S.\alpha'.\Delta\theta'}{2.r}$$

C. Railway Structure Design

Let's assume a fixed value of bridge variation temperature. As it's sown in figure (4), three cases may be studied separately:

- Bridge length L_1 is greater than l': in that case, the axial force may increase if Δθ' increases until that its abscissa value meets the bridge length;
- Bridge length L_2 is equal to l': in that case, the axial force diagram is on its maximum value;
- Bridge length L_3 is lower than l': in that case, the axial force shall be cut at the bridge length abscissa. Therefore, the additional loads on the rail are harnessed to not exceed that value.

In general, bridge length and its temperature variation are related to additional loads on the rail according to the equations:

$$L = \frac{5.E.S.\alpha'.\Delta\theta'}{2.r} \quad and \quad \Delta F = E.S.\alpha'.\Delta\theta'$$

Experience shows that:

$$\Delta\theta' = \frac{\Delta\theta}{2}$$

Where Δθ is rail temperature variation. Therefore, the total axial load on the rail may be written as:

$$\Delta F = \frac{3}{2}.E.S.\alpha.\Delta\theta$$

Apart from the bridge, the tolerable temperature variation on the rail has not to exceed 45°C. this value corresponds to a maximum bridge temperature variation equal to 15°C. In that case:

$$L < 108.3\,m \quad and \quad \sigma_R = 34.65\,MPa$$

Where σR is the additional stress on the rail.

Practically, bridge length is limited to 100 m in Morocco. In that case, the limit stress diagram is given in the figure (5) below. We assume that:

- Rail temperature variation is equal to 30°C;
- Bridge temperature variation is equal to 15°C.

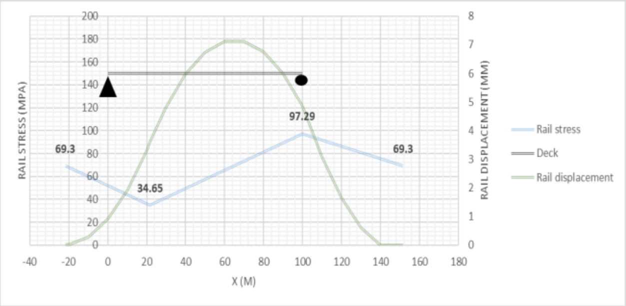

Fig. 5.Axial force distribution and rail displacement over an isostatic bridge (L = 100 m)

If the bridge length limit value is exceeded, dilatation devices have to be used. In fact, these devices are long (about 30 m) and need to be installed properly on a stable platform. Consequently, long railway bridges are designed in general by constructing intermediate isostatic decks of which length is about 40 m supposed to receive dilatation devices. Expansion joints may also be needed in order to retain ballast.

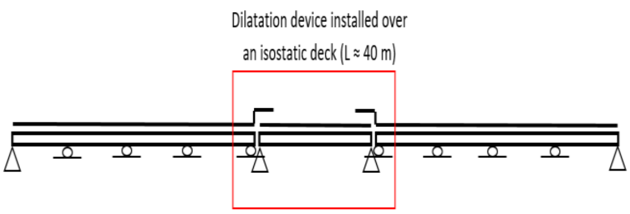

Fig. 6.Railway structure design for long continuous bridges

III. CASE STUDY: MHARHAR VIADUCT

This chapter aims to present an example of calculation, carried out on a long bridge where a non-compliance was noticed, in order to emphasize, on the basis of a real case, the track-bridge interaction phenomenon. Track geometry measurement is used to identify its instability. Only thermal actions are taken into account.

A. Description of viaduct

This is a long bridge of three continuous decks with lengths of 383.75m, 48m and 214.25m, respectively. The overall length is equal to 650m. The first deck is composed of seven spans, whereas the third deck contains 4 spans. Spans lengths are 50.625m and 56.500m.

The intermediate isostatic deck is constructed to receive a dilatation device. Expansion joints are also provided to retain ballast. Figure (7) shows a schematic diagram of the viaduct.

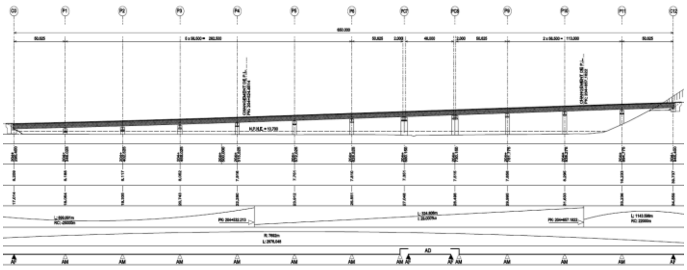

Fig. 7.Schematic diagram of the viaduct

It has been observed in the operational phase that the dilatation device, supposed to absorb rail dilatation from the two extreme parts of the bridge, is inverted. As a result, the end points of the rail are blocked, which cause additional stress on the rail and, consequentlty, a potential risk of buckling under axial compression.

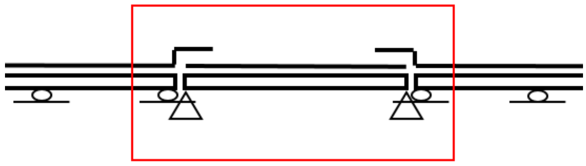

Fig. 8.Normal layout of the dilatation device

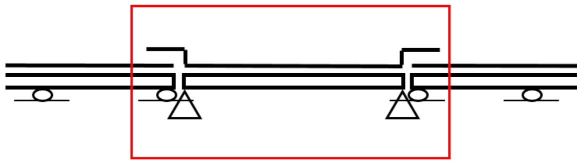

Fig. 9.Non-compliance observed

B. Track geometry measurement

Track unevenness is recorded in Morocco through a measuring car named EM120 (figure (8)). Since 2014, It has been equipped by an IMU (Inertial Measuring Unit) system that may provide an accurate recorded track geometry that fills the recommendations of EN 13848-2 standard.

Fig. 10. Moroccan geometry measuring car EM120

The parameters taken into account in this study are levelment and alignment.

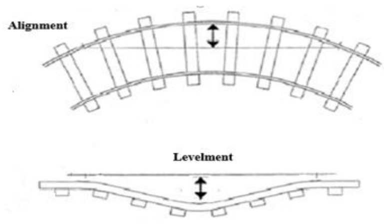

Fig. 11. Track geometry parameters

In order to higlight the effect of additional rail stress on track geometry, the measurement is filtered on the intermediate range wavelength which is equal, according to EN 13848-1 standard, to [25m – 70m].

C. Results and discussion

We assume the most unfavorable conditions of track exploitation. Therefore:

- Rail temperature variation is equal to 45 °C;
- Bridge temperature variation is equal to 35 °C.

The diagrams below show rail stress and rail displacement over the bridge.

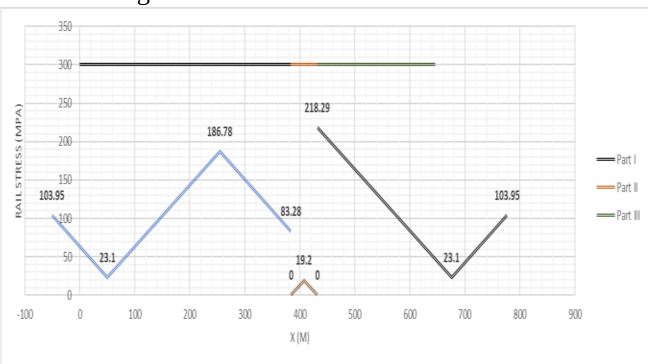

Fig. 12. Rail stress diagram

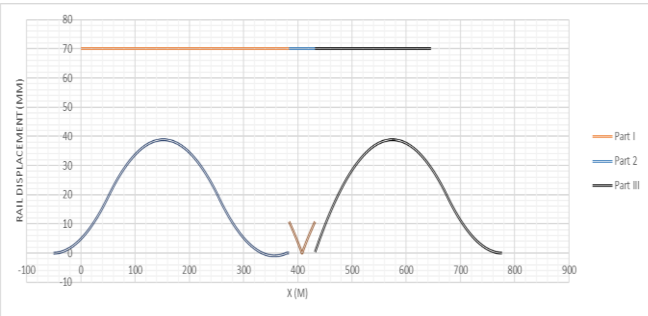

Fig. 13. Rail displacement diagram

We conclude that:

- As can be seen there is a maximum of compression stress at 186.8 MPa in the first deck and at 218.9 MPa in the third deck. Therefore, the limit value defined on UIC 774 leaflet is exceeded (72 MPa added to 103.95 MPa of confinement stress: 175.95 MPa);
- The maximum rail displacements are located at:
  - 232.16 m from dilatation device for the first deck;
  - 145.6 m from dilatation device for the second deck.

If we compare these results to track geometry measurement, we may obviously spot two points of levelment defects that correspond to maximum rail displacement locations.

In the other hand, we can also notice several alignment defects near the dilatation device caused mainly by a blocked CWR in his end.

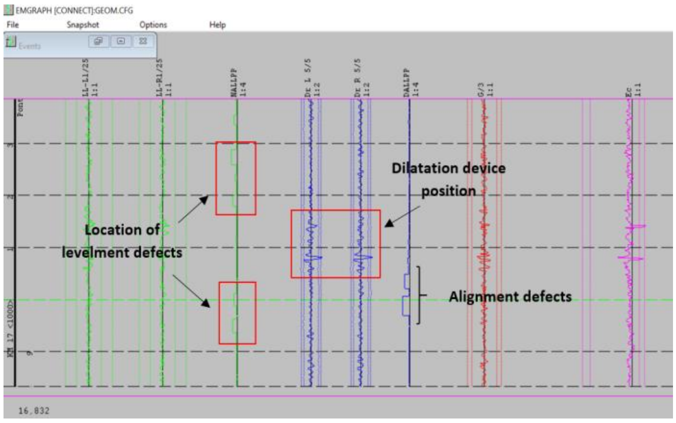

Fig. 14. Track geometry defects near the dilatation device

IV. CONCLUSION

The experience related to track-bridge interaction in Morocco is briefly introduced.

The Equations that govern the behavior of a CWR on a stable platform and over a bridge were established. As a result, a design guideline of railway structures was described.

As an application, a real case study is presented. We could conclude about the impact of track-bridge interaction on rail behavior by comparing the results to a real track measurement.

As a conclusion, we presented in this work a methodology that could be applied while conceiving a bridge supposed to receive a railway superstructure in order to avoid any track instability.

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