Stress on Ballast-Bed and Deterioration of Geometry in a Railway Track

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Abstract: In Greece extended cracking of twin-block concrete sleepers (ties) and fouling of the ballast-bed were observed with implied problems of gauge widening and deterioration of track’s geometry. This led to a ten-year investigation program, during which a new method was developed for the estimation of actions on track panel as well as of the pressures/stresses that develop under the seating surface of the sleeper on the ballast-bed. Results from the tests performed on the ballast used in the Greek network are also presented, conducted in laboratories in France, Austria, and Greece. The influence of the actions –static and mainly dynamic– on the track response and the stress and strain of the ballast-bed are also discussed as derived from the tests and theoretical analysis.

Key words: Railway ballast, ballast hardness, actions on track, mean stress on ballast bed, cracked concrete ties.

1. Introduction

The railway track is a vertical succession of various materials or layers of materials that define the final position of the rail running table as well as the properties of the track itself, as it “reacts” to the “action” that is created from the motion of the railway vehicle. Each material or layer that constitutes the line can be simulated by a combination of a spring with spring constant $\rho_i$ and a damper with damping coefficient $c_i$ (Fig. 1).

During the study for the dimensioning of a railway track and the selection of the individual materials constituting the track, it was found that the “weak links” are the ballast and the substructure underneath the superstructure. These are the elements of the track that develop residual deformations: deflections/subsidence and lateral displacements directly connected to the deterioration of the geometry of the track, which can be more specifically described as quality of the track. The smaller the residual deformations and their increase over time, the better the quality of the track. The ballast bed must ensure damping of the larger part of the vibrations of the train, adequate load distribution, and prompt drainage of rainwater. To fulfill these specifications it should not be “compacted”/fouled. In the Greek network, which is partly rehabilitated and partly newly constructed for maximum speed of 200÷250 km/h (=155.38 m/h) and axle load 22.5 t, during the 1970’s and the 1980’s, when the network was operated with maximum speed of 120 to 140 km/h (=74.58 to 87 m/h), cracks appeared on twin-block concrete sleepers with simultaneous fouling of the ballast bed. A ten-year investigation program was undertaken to study the causes of this phenomenon.

In the frame of this investigation, a new approach for the actions on sleepers and the ballast has been developed, that takes into account the real conditions of the track (maintenance etc.). In this paper the new method is described for the more accurate estimation of the acting loads on the track and the results of the tests performed on the ballast used in the Greek network are presented. Finally, the relation of the actions (static and dynamic) and the implied pressure on the ballast-bed and the deterioration of ballast are presented.

2. Investigation of the Ballast Quality

2.1 Background

In Greece until 1999, when the railway network
superstructure between Paleofarsala and Kalambaka began to be constructed, only twin-block concrete sleepers were used, which were of French technology, the majority of them being of Vagneux U2, U3 type with RN fastenings, and they presented extended cracking as described below. From that year on, only monoblock German technology B70 type of pre-stressed concrete sleepers with Vossloh W-14 fastenings are laid. Up to the end of the investigation program, for the system “sleeper–ballast”, only ballast of low hardness was laid on track-beds in Greece. The reason for this is that more than 65% of the mainland Greece is composed of limestone rocks. Thus, the supplying of non-limestone ballast should be imported from abroad or from certain regions of Greece (islands), both solutions of extremely high cost. After the appearance of cracks on approximately more than 60% of the total number of the U3 twin-block sleepers laid on the Greek network, as well as the observation of extended fouling of the ballast-bed, it was necessary to investigate the issue further and determine its causes. It should be underlined that, up to that time, the available relevant bibliography did not give any satisfactory justification of the phenomenon.

2.2 Research Team

A research program was initiated for the study of the “sleeper-ballast” system in Greek conditions (rolling stock, ballast quality, rail running table, level of maintenance, etc.). The research program (in which the author participated as head of the Hellenic railway scientific team and coordinator of the research) was conducted by OSE—Hellenic Railways Organization – in collaboration with Universities, Laboratories and Research Centers of Railway networks. The Research team consisted of: (a) the scientific team of engineers of the Track Directorate of OSE [1-2], (b) the National Technical University of Athens [3-4], (c) the Polytechnic School of the Aristotle University of Thessaloniki [5], (d) the Hellenic Ministry of Environment, Land Planning and Public Works/Central Public Works Laboratory, (e) the Hellenic Institute of Geological and Mineral Research [6], (f) the cooperation of the “Track” Research Department of the French Railways, (g) the University of Graz [7-8], (h) Transurb Consult, subsidiary of Belgian Railways (SNCB) etc. It included investigation of the phenomena that occurred on the track, laboratory tests and experiments on each individual material, with a view to identify the reasons for the systematic appearance of cracks on the sleepers. The research programs in France, in Aristotle University, and in the University of Graz focused on the quality and mechanical properties of the ballast laid on the Hellenic railway network. After the results of the investigation OSE, created its own laboratory in order to perform tests for ballast quality.
2.3 Laboratory Tests in France

In the frame of a technical cooperation between the French Railways (SNCF) and the Hellenic Railways Organization (OSE) / Track Directorate, a test program was performed in the Test Center of the “Direction de l’ Equipement” of the SNCF, to study the interaction between the ballast and different sleeper types. The scope of the experimental program was the simulation of the dynamic loads acting on the track in order to determine the influence of the sleeper’s type on the behavior of the ballast (limestone at that era) used in the Hellenic network [9]. The granulometric curve of the samples was according to the Greek regulations. In railways (a) the track must be completely compliant to the interoperability specifications (TSI’s) as in the State-members of E.U. and (b) the rolling stock must be completely compliant to the rules of the International Union Of Railways (U.I.C.) and to the regulations for Interoperability as in all countries – members of E.U.

2.3.1 Pulse Load Tests

Tests were performed in a cylindrical bucket from lamina (60 cm diameter, 250 cm height) with two different loading mechanisms: (i) using a steel plate with dimensions 31×31 cm², and (ii) using a steel plate with dimensions 34.5×34.5 cm², that is 1.25×(31×31) cm² (Fig. 2). The number of cycles of each test was 1,000,000 and the load was applied at a frequency of 4 Hz. The load was calculated so that the pressure under the plate would be correspondent to the Region R1 for the concrete sleepers strength [10], i.e., 120 kN.

For the U3 concrete twin-block sleepers there are three regions of “strength”: the R1 region begins on 125 ÷ 130 kN, the region R2 on 140 kN with the R3 region between 140 and 175 kN. The base of the bucket was replaced with a “pad” made of synthetic felt. Only the ballast particles not passing through the 25 mm sieve were chosen and the weight of the ballast, which was placed in the bucket, was measured. The subsidence was measured every 250,000 cycles, with the first measurement performed after 5,000 cycles. These tests were performed with the application of a pulse load in the laboratories of SNCF (Fig. 2). Quality tests were also performed on Vagneux U31 twin-block concrete sleepers (produced according to the technical specifications of the SNCF in Thessaloniki, Greece and sent from Greece to France), and two sleepers, one twin-block Vagneux U41 type and one monoblock, manufactured in France. The tests were performed under a cyclic load between 10 and 59 kN. The description and the results for the pulse load test are presented in Ref. [11].

2.3.2 Cyclic Load Tests

The device used for this type of tests is a unidirectional vibrator, which applies cyclic load on the sleeper between ±80 kN and 2.50 kN at a frequency of 50 Hz. The duration of each test was 50 hours and only the ballast particles not passing through the 25 mm sieve were tested. After staying 100 hours in the device the ballast was removed and the ballast particles not passing through the 25 mm sieve as well as the crushed ballast (during the test) that passed through the 1.6 mm sieve was weighed. Tests were also performed
on: (a) 1 twin-block concrete sleeper type Vagneux U31 produced in Greece, (b) 1 twin-block concrete sleeper type Vagneux U41 produced in France, (c) 1 wooden sleeper, (d) 1 monoblock sleeper of prestressed concrete produced in France, (e) 1 twin-block concrete sleeper type Vagneux U2 produced in Greece.

The ballast wear seems to be of less importance in the case of the wooden sleeper, even though the amount of fractured ballast particles is almost the same as in the case of a U41 sleeper or a monoblock sleeper. As far as the abrasion is concerned, the U31 sleeper produced in Thessaloniki presented a worse behavior than the sleepers produced in France. Very similar to that is the behavior of the U2 sleeper with octagonal blocks produced in Greece. The U41 sleeper yielded very good results, although the deterioration of the ballast is worse than that of the monoblock sleeper. In more detail the results for the cyclic load tests are cited in Ref. [11].

2.4 Laboratory Tests in Austria

Ballast material and track components were sent to the University of Graz to be tested. The investigation included several tests, which are presented in the following paragraphs.

2.4.1 Tests of Ballast Quality

These tests were conducted according to (a) the regulations valid in Austria, i.e., conditions of ballast delivery, ÖBB, BH 700, petrographic evaluation ÖNORM B 3120, pressure strength ÖNORM B 3124, crushing resistance ÖNORM B 3127 / A / Pkt 2 and (b) the regulations of the Hellenic Railways Organisation, Los-Angeles-Test NF P18-573, Oct. 1978, Deval-Test NF P18-577, April 1979, determination of DRi-value, according to “Specification Technique 695E” of SNCF, 25-10-1984 (as in Annex 1).

2.4.2 Repeated Load Test

This test was executed in the “Ballast-bed-Simulator” of the Institute of Rail Transport, Technical University Graz. The set-up permitted to record: development of settlement, elastic properties, and pressure on the soil level below ballast, ballast gradation underneath sleeper-center before and after the test procedure. The parameter variation was agreed as follows: ballast depth 25 and 35 cm, 3 types of sleeper: OSE twin-block concrete sleeper U31, OSE Azobe timber sleeper, ÖBB monobloc concrete sleeper L1. The types of sleepers tested are summarized in Ref. [11]. It was found appropriate to arrange the individual test per parameter combination as follows:

- 100,000 cycles: 20 ÷ 60 kN per sleeper
- $1.5 \times 10^6$ cycles: 20 ÷ 120 kN per sleeper
- $0.5 \times 10^6$ cycles: 20 ÷ 180 kN per sleeper

with a load-frequency of 7 Hz.

The load range of the lab tests was representative of the load conditions on track [8]. The results of the Deval tests and the Los Angeles tests are presented in [11]. The calculation of the coefficient of durability DRi according to the French specifications, which were in effect at that time, is cited in Annex 1. These values should be compared to minimum values of the French Regulations (see Table 2)–since the sleepers and the track were constructed under the French regulations–given in Table 2. From the above it is derived that the ballast source material should possess certain characteristics in order to achieve reasonable uniform quality-properties [8]. The results of the tests cited above in Table 1 proved that the ballast laid on tracks of the Greek railway network was of good quality, according to the Greek (and French) regulations, so a further investigation was not needed. For such quality the parameters cited below in the analysis could be used.

2.4.3 Repeated Load-tests in the Ballast-bed Simulator

The development of the permanent settlement was recorded at a number of intermediate stops during the

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Coeff. DEVAL SE</th>
<th>Coeff. ANGELES - LA</th>
<th>DRi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14.14</td>
<td>25.98</td>
<td>13</td>
</tr>
<tr>
<td>2</td>
<td>14.89</td>
<td>24.26</td>
<td>14</td>
</tr>
<tr>
<td>3</td>
<td>15.55</td>
<td>24.09</td>
<td>15</td>
</tr>
<tr>
<td>4</td>
<td>15.38</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mean Value</td>
<td>14.99</td>
<td>24.78</td>
<td>14</td>
</tr>
</tbody>
</table>
Repeated-Load-Tests. The measurements were taken without vertical load and are therefore comparative with each other. For ballast depth of 25 cm, all sleepers display a loss of cross level by reasonable differential settlements of the sleeper-ends. For ballast depth of 35 cm, again the differential settlement is evident. In more detail the results for the cyclic load tests are cited in Ref. [11].

2.4.4 Crushing Resistance Test

This test was executed according to the ÖBB-specification B700 [8] by the Technical Test and Research Centre, TU Graz. 2.11 t of ballast were placed in a container in layers and loaded by a piston with 400 kN. The evaluation was done by using the calculation-scheme given in the standard above. While one test just missed the minimum value of W=55, the two other probes came out above this limit (see Annex 2). A thorough evaluation of the tests in Austria is cited in Ref. [12].

2.5 Laboratory Tests in Greece

2.5.1 Laboratory and In-situ Tests

Several research programs funded by OSE, concerning ballast and sleepers, were executed in Greece. Among them we cite here the following regarding: (a) the mechanical behavior of ballast in Aristotle’s University of Thessaloniki [5], (b) concrete twin-block sleeper behavior in the National Technical University of Athens [4], (c) monoblock sleepers of prestressed concrete behavior [3], (d) ballast sources [6] and (e) CBR tests [8]. The results of the research program for the mechanical behavior of ballast [5] performed at Aristotle University of Thessaloniki are presented in Ref. [11], and presents the results of the durability of ballast from different sources (quarries) from all over Greece. In more detail the results for the cyclic load tests are cited in Ref. [11].

2.5.2 CBR Field Test

The results of the CBR-field-tests, taken at three representative positions along the Athens- Thessaloniki line, according to their characteristics, are listed in Table 3. It is noted that the quality of the subgrade can be determined from the CRB tests according to the following ranges:

- “good subgrade” 15-40%
- “excellent subgrade” 40-100%

The results indicate a very good quality subgrade, which in few cases tends to imply a somewhat too stiff soil. This quality of subgrade -as in Table 3- complies with the range of $\rho_{\text{subgrade}}$ used in the analysis below.

### Table 3  CBR results [12].

<table>
<thead>
<tr>
<th>Position</th>
<th>CBR 2.50 mm</th>
<th>CBR 5.00 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>19.56%</td>
<td>16.11%</td>
</tr>
<tr>
<td></td>
<td>43.72%</td>
<td>35.28%</td>
</tr>
<tr>
<td></td>
<td>17.03%</td>
<td>15.95%</td>
</tr>
<tr>
<td>B</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>69.03%</td>
<td>107.38%</td>
</tr>
<tr>
<td></td>
<td>44.87%</td>
<td>50.62%</td>
</tr>
<tr>
<td></td>
<td>77.08%</td>
<td>92.04%</td>
</tr>
<tr>
<td></td>
<td>44.87%</td>
<td>74.40%</td>
</tr>
<tr>
<td></td>
<td>35.66%</td>
<td>40.65%</td>
</tr>
<tr>
<td>C</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>20.71%</td>
<td>20.94%</td>
</tr>
<tr>
<td></td>
<td>32.31%</td>
<td>33.75%</td>
</tr>
<tr>
<td></td>
<td>39.12%</td>
<td>57.52%</td>
</tr>
<tr>
<td></td>
<td>25.31%</td>
<td>23.01%</td>
</tr>
<tr>
<td></td>
<td>35.66%</td>
<td>48.32%</td>
</tr>
<tr>
<td></td>
<td>62.12%</td>
<td>61.36%</td>
</tr>
</tbody>
</table>
3. Estimation of the Actions on Railway Track

3.1 General

The analysis is conducted by considering the rail to be a continuously supported beam on an elastic foundation consisting of closely spaced springs [13]. The deformation of the track due to bending from the vehicles (loads) circulation is depicted in Fig. 3, as well as the influence curves of several parameters [14].

\[ \frac{1}{\rho_{\text{total}}} = \sum_{i=1}^{n} \frac{1}{\rho_{i}} \]  

(1)

where \( \rho_{i} \) is the coefficient of “Rail Support Modulus” of each layer. This implies that \( \rho_{\text{total}} \) is a coefficient of quasi elasticity (stiffness) of the track, which is the “spring constant” of Hooke’s law. It is defined as the “reaction coefficient of the tie”, and \( \rho_{i} \) is the “spring constant” of each layer. In the aforementioned simulation, all the layers underneath the ballast-bed in the case of ballasted track (blanket-layer, subgrade, prepared subgrade, soil) are modeled as a totality through the coefficient \( \rho_{\text{substructure}} \). Moreover, the functions \( \eta(x) \) and \( \mu(x) \), the elastic length \( L \) (Fig. 3), moment \( M \), deflection \( y \), reaction/action \( R \) on the track per sleeper etc., are functions of \( \rho_{\text{total}} \) [14]:

\[ \eta(x) = e^{-Kx} \cdot \left[ \cos(Kx) + \sin(Kx) \right] \]
\[ \mu(x) = e^{-Kx} \cdot \left[ \cos(Kx) - \sin(Kx) \right] \]  

(2)

where \( E, J \) moment of inertia and modulus of elasticity of the rail, \( \ell \) the distance between the ties, \( y_{\text{max}} \) and \( M_{\text{max}} \) at the position \( x=0 \),

\[ K = \frac{1}{L} = \frac{\rho_{\text{total}}}{4EJ\ell} \]  

(3)

The ballast grain crushing (fouling) is directly correlated to the forces imposed by the passing of the axles of trains, transferred–via increasing contact surfaces—to the ballast-bed. The existing methods for the calculation of the actions/reactions \( R \) per sleeper on a railway track (in 1980’s), which are found in the German bibliography, the French bibliography, and AREMA, failed in predicting the systematic appearance of cracks in a large percentage of concrete twin-block sleepers in the Greek railway network, as well as the influence of the very poor condition of the ballast-bed. This led to the development of a new method [14-15] for the estimation of the actions on track that successfully predicted and explained the observed conditions.

3.2 Actions on Track Panel According to German Literature

In the German bibliography, the total load \( Q_{\text{total}} \)
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(static and dynamic) acting on the track, is equal to the static wheel load multiplied by a factor. After the total load is estimated, the reaction $R$ acting on a sleeper, which is a percentage of the total load $Q_{total}$ can be calculated [16-17]:

$$Q_{total} = Q_{wh} \cdot (1 + t \cdot s)$$  \hspace{1cm} (5)

where: $Q_{wh}$ is the static load of the wheel,
$s = 0.1 \cdot \varphi$ to $0.3 \cdot \varphi$ dependent on the condition of the track, that is:
$s = 0.1 \cdot \varphi$ for excellent track condition
$s = 0.2 \cdot \varphi$ for good track condition
$s = 0.3 \cdot \varphi$ for poor track condition
and $\varphi$ is determined by the following formulas as a function of the speed:
For $V < 60$ km/h then $\varphi = 1$.
For $60 < V < 200$ km/h then:
$$\varphi = 1 + \frac{V - 60}{140}$$  \hspace{1cm} (6)

where $V$ the maximum speed on a section of track and $t$ coefficient dependent on the probabilistic certainty $P$ ($t = 1$ for $P = 68.3\%$, $t = 2$ for $P = 95.5\%$ and $t = 3$ for $P = 99.7\%$).

The reaction $R$ of each sleeper is calculated according to [16]:

$$R = \frac{Q_{total} \cdot \ell}{2 \cdot L} = \frac{Q_{total} \cdot \ell}{2} \cdot \frac{1}{\sqrt{\frac{4 \cdot E \cdot J}{b \cdot C} \cdot \rho_{total}}}$$  \hspace{1cm} (7)

$$= \frac{Q_{total} \cdot \ell}{2} \cdot \frac{1}{\sqrt{\frac{4 \cdot E \cdot J}{b \cdot C} \cdot \rho_{total}}} = A_{stat} \cdot Q_{total}$$

where $\ell$ = distance among the sleepers
$L = \sqrt{\frac{4 \cdot E \cdot J}{b \cdot C}} = \sqrt{\frac{4 \cdot E \cdot J}{\rho_{total}}}$  \hspace{1cm} (8)

$C =$ ballast modulus [N/mm$^3$]

$b =$ a “width of conceptualized longitudinal support” according to Zimmermann that multiplied by $\ell$ equals to the loaded surface $F$ of the seating surface of the sleeper

3.3 Actions on Track Panel According to French Literature

According to the French bibliography

$$Q_{total\ max} = Q_{wh} + \sigma_{\alpha} + 2\sqrt{[\sigma^2(\Delta Q_{NSM})] + [\sigma^2(\Delta Q_{SM})]}$$  \hspace{1cm} (9)

where: $Q_{wh}$ = the static load of the wheel
$Q_{\alpha}$ = semi-static load due to the cant (superelevation) deficiency

$\sigma(\Delta Q_{NSM})$ is the standard deviation of the dynamic component of the total load due to Non Suspended Masses (NSM) that participates in the increase of the static load (see Fig. 4).

$\sigma(\Delta Q_{SM})$ is the standard deviation of the dynamic component of the total load due to the Suspended Masses (SM) that participates in the increase of the static load (see Fig. 4).

The reaction/action on the sleeper is given by [17-18]:

$$R_{total\ max} = \left[ Q_{wh} \cdot \frac{1 + \frac{Q_{\alpha}}{Q_{wh}}}{140} \right] + 2\cdot \sqrt{[\sigma^2(\Delta Q_{NSM})] + [\sigma^2(\Delta Q_{SM})]} \cdot A_{stat} \cdot 1.35$$  \hspace{1cm} (10)

Where: $A_{stat} = \frac{1}{2\sqrt{2}} \cdot \frac{\ell^3 \cdot \rho_{total}}{E \cdot J}$  \hspace{1cm} (11)

$\rho_{total}$ = the total static stiffness (elasticity) of the track

![Fig. 4 Vehicle - Track as a Combination of Springs and Dampers [21]. The Suspended Masses of the Vehicle are over the primary suspension of the Bogie, the Non-Suspended Masses are below the primary suspension of the Bogie (track’s section included), as in the shaded area.](image)
3.4 Proposed Method for Actions on Track Panel (Giannakos 2004)

Until the investigation program there were three theories for the calculation of loads. All are giving results for sporadic cracks in sleepers laid on track, since an extensive cracking was re-marked in a percentage of 60%. We developed a method verifying these results derived from the experience on track [14]. The actions on track panel are calculated with the following equation:

\[
R'_2 = \left( Q_{\text{wh}} + Q_{\alpha} \right) \cdot \tilde{A}_{\text{dyn}} + \\
\mu \cdot \sigma \left( \Delta R_{\text{NSM}} \right) + \sigma \left( \Delta R_{\text{SM}} \right)
\]

(12)

where \( Q_{\text{wh}} \) = the static wheel load, \( Q_{\alpha} \) = the load due to cant deficiency, \( \tilde{A}_{\text{dyn}} \) = dynamic coefficient of sleeper’s reaction, \( \mu \) = coefficient of dynamic load (3 for a probability 99.7 % and 5 for 99.9 %), \( \sigma(\Delta R_{\text{NSM}}) \) = the standard deviation of the dynamic load due to non suspended masses (see also [21] for the track mass included), \( \sigma(\Delta R_{\text{SM}}) \) = the standard deviation of the dynamic load due to suspended masses and :

\[
\tilde{A}_{\text{dyn}} = \frac{1}{2} \sqrt{2} \sqrt{\frac{E \cdot J}{h_{\text{TR}}}}
\]

(13)

where \( h_{\text{TR}} \) the total dynamic stiffness of the track given by:

\[
h_{\text{TR}} = \frac{1}{2} \sqrt{2} \sqrt{\frac{E \cdot J \cdot \rho_{\text{total}}}{\ell}}
\]

(14)

The interested reader should see Refs. [14-15] about \( \tilde{A}_{\text{dyn}} \) and \( h_{\text{TR}} \). Since the vehicle’s motion is a random dynamic phenomenon, this implies that the dynamic co-efficient \( \tilde{A}_{\text{dyn}} \) should be used instead the static one \( \tilde{A}_{\text{stat}} \) contrary to the international literature. The results from this method predict an extended appearance of cracks in twin-block sleepers that was indeed observed on the Greek permanent way (60% or even higher). In contrast, the German, French and American methods give results predicting no cracks or sporadic cracking in the order of 1–2%, that are not compatible with the real conditions. In Ref. [20] this method is taken into account for precast concrete rail-way track systems.

3.5 Actions on Track Panel According to American Literature

For comparison, the method in American bibliography is cited. This method is described in Refs. [13, 21] and it is based on the same theoretical analysis of continuous beam on elastic foundation. The total load (static and dynamic) acting on the track is dependent on an impact factor \( \theta \):

\[
\theta = \frac{D_{33} \cdot V}{D_{\text{wheel}} \cdot 100}
\]

(15)

where: \( D_{33} \) the diameter of a wheel of 33 inches
\( D_{\text{wheel}} \) in inches also the wheel’s diameter of the vehicle examined
\( V \) the speed in miles / hour

The total load is:

\[
Q_{\text{total}} = Q_{\text{wheel}} \cdot (1 + \theta)
\]

(16)

The maximum deflection and moment are [13]:

\[
y_{\text{max}} = w(0) = \frac{\beta \cdot Q_{\text{total}}}{2 \cdot U}
\]

(17)

\[
M_{\text{max}} = M(0) = \frac{Q_{\text{total}}}{4 \cdot \beta}
\]

(18)

where: \( U \) in lb/inch\(^2\) is the rail support modulus derived by the relation \( p = U \cdot y \) and as easily can be found \( U = \rho / \ell \) [13] and

\[
\beta = \frac{\sqrt{U}}{4 \cdot E \cdot J} = \frac{\sqrt{\rho_{\text{total}}}}{4 \cdot E \cdot J \cdot \ell} = \frac{1}{L}
\]

(19)

\( L \) is the “elastic length” given previously by the Eq. (8). The influence curve for \( y \) (that is for deflection) given in Ref. [13] is used to determine the largest value \( p_{\text{max}} \) and the maximum rail seat load \( R_{\text{max}} \) on an individual tie is given by:

\[
R_{\text{max}} = p_{\text{max}} \cdot \ell = U \cdot y_{\text{max}} \cdot \ell
\]

\[
= U \cdot y_{\text{max}} \cdot \ell = U \cdot \frac{\beta \cdot Q_{\text{total}}}{2 \cdot U} \cdot \ell =
\]

\[
= \frac{4 \cdot \rho_{\text{total}} \cdot Q_{\text{total}}}{2} \cdot \ell =
\]

\[
= \frac{1}{2} \sqrt{2} \sqrt{\frac{E \cdot J}{4 \cdot E \cdot J \cdot \ell}} \cdot Q_{\text{total}} = \tilde{A}_{\text{stat}} \cdot Q_{\text{total}}
\]

(20)
where $Q_{\text{total}}$ is calculated from Eq. (16) and $A_{\text{stat}}$ is the same as in Eq. (11).

3.6 Estimation of Actions on Track by the Four Methods in the Case of Cracked Sleepers/ties

For the calculations of the values in Fig. 5 the following data were used (about the values of the coefficients $\rho_i$ for rail, concrete tie, ballast and substructure, according to measurements of the German Railways (DB) see Ref. [14]):

- Rail UIC54, 54 kg/m, $J = 2.346 \times 10^7 \text{ mm}^4$, $\rho = 75000 \text{ kN/mm}$.
- Twin-block concrete tie U2/U3, seating surface $185800 \text{ mm}^2$, $\rho = 13500 \text{ kN/mm}$.
- Ballast 30 cm thickness, two years after laying, $\rho = 380 \text{ kN/mm}$.

Fastening RN with rail pad of 4.5 mm or fastening W14 with rail pad Zw700, $\rho$ derived from the load-deflection curve. To calculate the real acting forces on the superstructure and the ties, applying the above-mentioned equations, in a multi-layered construction with poly-parametrical function, the exact rigidity of the elastic pad of the fastening for each combination of parameters must be determined. In the case of the RN fastening we must find and use the tie-pad stiffness of the 4.5 mm pad, according to its load-deflection curve. The most adverse curve is used because it describes the behavior of the pad during the approach of the wheel since the second curve describes the unloading of the pad after the removal of the wheel. The stiffness of the substructure varies from 40 kN/mm for muddy substructure to 250 kN/mm for rocky tunnel bottom with not enough ballast thickness. Each time this stiffness changes in the equations above, the “acting” stiffness of the tie-pad also changes and it is derived after an equilibrium of the “springs” calculated by the trial-and-error method.

- $\rho_{\text{substr}}$ varies from 40 kN/mm for muddy substructure to 250 kN/mm for rocky tunnel bottom with not enough ballast thickness.
- maximum speed 140 km/h or 87.01 mi/h, axle load 22.5 t or 49604 lb, NSM=2.54 t or 5599.74 lb, normal gauge 1435 mm, height of center of gravity of the vehicle from the rail running table 1500 mm. For more details about NSM calculations the interested reader should see Refs. [14-15].

- Coefficient $k$ of the condition of the rail running table is used for two discrete values 9 (average value of a non-ground rail running table) and 12 (maximum acceptable value for non-ground rail running table for speed $\geq 140 \text{ km/h}$ or 87.01 mi/h). For more details the interested reader should see Ref. [14-15].

- The damping coefficients do not influence at all the distribution of loads across each rail, as the total of international literature cites. It can only influence the track mass participating in the motion of the Non-Suspended Masses of the vehicle (for more details see Ref. [22]).

For the U2/U3 concrete twin-block sleepers, which presented extended cracking, there are three regions of “strength”: R1 region begins at 125–130 kN, R2 region (cracking threshold) at 140 kN and the R3 region (failure threshold) between 140 and 175 kN. Fig. 5 depicts the results derived from the four aforementioned methods. The force on a concrete sleeper U2/U3 laid on the Greek network is calculated for two types of fastenings RN (medium stiff) and W14 of VFS (very resilient), for two values of rail running table coefficients, $k = 9$ and 12. The force is plotted against the stiffness coefficient of the substructure. The forces on the tie are calculated according to the French, the German, the AREMA, and Giannakos (2004) method. On the same figure the limits of the two aforementioned regions of “strength” R2 and R3 are plotted for the U2/U3 tie as described in its technical specifications. The characteristic maximum value for $\rho_{\text{substr}}$ is 100 kN/mm. It is noted that the loads (for $\rho_{\text{substr}} = 100$) on the tie, estimated by the AREMA, the French, and the German methods, are below the R2 Region limit (125–130 kN). This means that no cracking of the ties is predicted with these three methods, in contrast with the situation observed on track in the Greek
Fig. 5a Results for twin-block concrete tie U2/U3 and RN fastening with 4.5 mm tie pad and k=12 (non ground rail running table just before grinding) according to: (a) the method from the German bibliography Eq. (7), (b) the method from French bibliography Eq. (10) (c) Eq. (20) - AREMA method and (d) Giannakos (2004) method Eq. (12) with $\nu=3$. Region 2 is the Cracking Threshold (Between 125 and 130 kN) and Region 3 is the Failure Threshold (between 140 and 175 kN) for the twin-block U2/U3 Concrete Sleepers.

Fig. 5b Results for twin-block concrete tie U2/U3 and RN fastening with 4.5 mm tie pad and k=9 (non ground rail running table medium condition) according to: (a) the method from the German bibliography Eq. (7), (b) the method from French bibliography Eq. (10) (c) Eq. (20) - AREMA method and (d) Giannakos (2004) method Eq. (12) with $\nu=3$. Region 2 is the Cracking Threshold (Between 125 and 130 kN) and Region 3 is the Failure Threshold (between 140 and 175 kN) for the twin-block U2/U3 Concrete Sleepers.
Fig. 5c  Results for twin-block concrete tie U2/U3 and W14 fastening with clip Skl14 and Zw700 tie pad and k=9 (non ground rail running table medium condition) according to: (a) the method from the German bibliography Eq. (7), (b) the method from French bibliography Eq. (10), (c) Eq. (20) - AREMA method and (d) Giannakos (2004) method Eq. (12) with \( \nu = 3 \). Region 2 is the Cracking Threshold (Between 125 and 130 kN) and Region 3 is the Failure Threshold (between 140 and 175 kN) for the twin-block U2/U3 Concrete Sleepers.

Railway Network with appearance of extended cracking (over of 60\%) of the ties laid on track. On the other hand, Giannakos (2004) method estimates load levels on the ties that lie within the R3 Region and is successful in predicting the extended cracking that was observed. The more interested reader can refer to Refs. [10, 15]. This leads to the adoption of this method (Giannakos) as more accurate for the simulation of the real–on track–conditions, as far as the acting loads on ties. International Federation of Concrete (fib) adopted this method as the sole reference in Ref. [20] for “Precast concrete railway track systems”, between them concrete ties are included.

4. Mean Stress on Ballast-Bed

Regarding the issue of ballast fatigue, the existing bibliography assumes a uniform distribution of stresses under the sleeper and without further details uses the mean value of stress. Nevertheless, various researches (UIC, SNCF, OSE) have questioned whether the mean value of pressure (stress) is representative. Based on bibliography, the maximum moment measured actually on track results from parabolic stress distribution [23]. But in reality, the seating of the sleepers is supported on discrete points (points of contact with the grains of the ballast) as Fig. 6 depicts and the resulting necessity to calculate the stress per grain of ballast cannot give comparative results to the rest of the bibliography [24]. So it is possible to use the mean value of stress not as an absolute quantity, but comparatively as a “criterion of judgment” and in combination with the possibility it covers [1-2].

There is no uniform support of the sleeper on the ballast, or uniform compaction of the ballast and the ground, and there are faults on the rail running table, imperfections on the wheels etc. Nevertheless the use of the mean pressure (stress) on ballast-bed should be used as a qualitative index– “judgment criterion”–to compare different tracks and make decisions. Since the publication of ORE [25] research (of the International Union of Railways [U.I.C.]), it has been established that the material of the sleepers (wood, concrete) gives almost identical values of settlement of the track. The residual settlement–that is the deterioration of the track’s
Stress on Ballast-Bed and Deterioration of Geometry in a Railway Track

Fig. 6  Ballast grains in the ballast bed and transmission of stresses and actions.

geometry—is a percentage of the total subsidence during the passing of the loads [21]. This implies that the performance of the track and especially of the ballast-bed—in terms of deterioration of the track’s geometry—should be extrapolated even if it is almost identical in all cases (wood, concrete). This fact is confirmed by more recent publications [20, 26].

This experimental confirmation, which has been also verified through calculations [1-2], means that in relation to the sustaining of the geometry of the track, the material of the sleeper has no significant influence. We will observe the same frequency of maintenance interventions to the sleeper-ballast contact surface, whether a wooden sleeper or concrete sleeper is used. The above experimental data have been verified with analytical results, through the calculation of the reaction of the sleeper and that of the mean value of stress $p$ and the subsidence $y$ for four types of sleepers.

Using the actions/reactions—due to static and dynamic loads—as depicted in Fig. 5, the mean stress on ballast-bed $\bar{p}$ is calculated. The mean stress $p_{ballast}$ on ballast-bed exerted by the tie’s seating surface should be calculated in order to be used as a selection/“judgment” criterion, according to the following equation:

$$\bar{p}_{ballast} = \frac{R_{tie}}{L_{eff-tie} \cdot b_{tie}} = \frac{R_{tie}}{(L_{tie} - e) \cdot b_{tie}}$$  \hspace{1cm} (21)

Where: $R_{tie}$ the reaction/action on each tie derived from each method

$L_{tie}$ = length of the tie, i.e., 8”-6’ or 2590 mm
$e$ = gauge of the track (~1500 mm)

$L_{eff-tie}$ calculated from Eq. (22)

$$L_{eff-tie} = (L_{tie} - e)$$  \hspace{1cm} (22)

The values of the average pressure according to each method—as cited above—are depicted in Fig. 7. The calculated average pressure $\bar{p}$ is much higher than the permissible stress 0.30 MPa on the ballast bed [27]. In some cases it is more than 300% higher. Even in the most characteristic case of substructure with $p_{substr}=100$ kN/mm its value (calculated by all methods) is extremely high. The calculation predicts the degradation of ballast that was observed in practice and the deterioration of the geometry of the track that was implied. Even in the case of wooden ties in spite of the greater seating surface of a wooden sleeper (approximately 13% greater than that of the U41 and 55% greater than U2/U3), it bears about 3% higher pressure in undeflected seating because of the different elastic pad (see Table 10 in Ref. [11]). Heavier concrete sleepers, in relation to the wooden ones, hinder the settlement of the track that is caused by vibrations [10]. With those sleepers no peaks are observed, which characterize the amplitude of vibration in the resonance area, and whose creation leads to destabilization of the ballast. Moreover, the reduction of the participating Non Suspended Masses in the system’s motion and the use of a “more elastic” pad, i.e., pad with small $\rho_{pad}$ ($\rho_{pad} < 100$ kN/mm and /or 80 kN/mm), leads to a reduction of the stressing of the ballast [11].

Using the AASHTO Road-Test equation:

Decrease in track geometry quality = (increase in stress on the ballast bed)$^m$ \hspace{1cm} (23)

where $m= 3$ to 4, the deterioration of the ballast-bed and the track’s geometry can be estimated in cost terms. As stress on the ballast bed the values from Fig. 7 should be used.

5. Conclusions

A new method for the estimation of actions [14] on track panel is presented and applied to estimate the mean pressure under the seating surface of sleeper on
Stress on Ballast-Bed and Deterioration of Geometry in a Railway Track

The ballast-bed of a track. The method is successful in predicting the situation observed on track, i.e. the appearance of extended cracking of twin-block concrete ties (U2/U3 type), the degradation of ballast and the resultant deterioration of the track’s geometry. Laboratory tests on low hardness ballast of the Greek railway network performed in Greece, Austria and France are presented and used to evaluate the situation. Research investigations and lab experiments verify that the track subsidence is independent of the sleeper material (wood, concrete). The proposed method –verified in practice- can safely be used to model the behavior of the ballast-sleeper system in relation to the rest of intervening parameters of vehicle and track (Non Suspended Masses, rolling roughness of rail running surface, use of resilient fastenings etc.). The method also provides a quantifying reasoning of the real situation observed on track.

Fig. 7  Mean stress $\bar{p}$ exerted by the tie’s U2/U3 seating surface on ballast-bed, as a “judgement” criterion.

References

ANNEX 1

DETERMINATION OF HARDNESS COEFFICIENT DRI/DRG
FROM THE COEFFICIENTS “LOS ANGELES” AND “DEVAL WET”

Fig. 1-1  Chart of ballast hardness (DRI) calculation, from Deval wet coefficient (horizontal axis) and Los Angeles coefficient (vertical axis) according to the Greek (and French) regulations.
## ANNEX 2

### Evaluation according to B "crushing resistance of track-ballast"

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Degree of destruction Z=1.316 pressure resistance w=54.90

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Degree of destruction Z=1.265 pressure resistance w=57.20

### degree of destruction pressure resistance

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The average value indicates the acceptance of the ballast in its crushing-resistance-properties.

**Fig. 2-1 Results of Crushing Resistance Test of Ballast [8].**