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Damage of Railway Sleepers under Dynamic Loads: A Case History from the Greek Railway Network

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DAMAGE OF RAILWAY SLEEPERS UNDER DYNAMIC LOADS: A CASE HISTORY FROM THE GREEK RAILWAY NETWORK

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ABSTRACT

In Greece, during the 1980's, 60% of the twin-block concrete ties designed for 200 km/h, which were laid on a track with maximum operational speed of 140 km/h, presented serious cracks. The existing theories could not justify the appearance of these cracks since the calculated actions on ties were much lower than the limit values. It was found that the geotechnical conditions of the track substructure played a key role in the damage of the sleepers. In this paper a model for the determination of the load acting on the track's superstructure is presented properly taking into account the geotechnical conditions of the track substructure. The basic parameters of concrete tie design considering the most adverse conditions of a railway network are investigated, and a methodology for calculating the load undertaken by each tie is proposed. Finally, numerical applications on twin-block and monoblock ties are presented, including the use of high resilience fastenings. Moreover an application for the heavy-haul rail transport is presented, in case of a track equipped with W24 fastening and concrete sleepers.

INTRODUCTION: THE GREEK RAILWAY NETWORK CASE HISTORY

In Greece, during the period of 1972 until 2000, only concrete twin-block ties (also called sleepers in European terminology) were placed on OSE (where OSE are the initials in latin for the Hellenic Railways Organization) tracks in operation (Fig. 1). The three types of reinforced concrete twin-block ties, Vagneux U2, U3 with RN fastenings, and U31 with Nabla fastenings –all of them of French technology– that were used in the Greek network are similar to those used in the same period by the French Railways (SNCF). These ties are laid on the French network with operational speed 200 km/hr and daily tonnage 50,000 t/day (Giannakos et al., 1990, 1991), whereas in Greece until the beginning of 2000, the maximum operational speed was 120÷140 km/hr (it is now ≥160 km/hr) and the daily tonnage did not exceed 10,000 t/day. Of the above types, 60% of the U3 ties exhibited cracks in the Greek network, at a position under the rail from the lower bearing area of the tie propagating upwards to the rail seat (Giannakos et al 1990, Ambakoumkin et al, 1992 - 1993). It is noted that the same tie type in the French Railways network did not exhibit any problems at all (Giannakos 2004).

According to the French regulations and Technical Specifications, tie U3 has a service load of 125 kN to 130 kN, design load 140kN to 175 kN and tie failure (nominal) load 175 to 200 kN (Prud' homme et al., 1976). Experimental tests, performed at the SNCF (initials for the French Railways Organisation) laboratories (Giannakos 2004) as well as at the National Technical University of Athens (NTUA) laboratories (Tasios et al, 1989), confirmed that the U3 OSE (latin initials for the Hellenic Raiways Organisation) ties fulfilled the requirements of 125 to 130 kN and 140 to 175 kN service and design load, respectively.

The existing international bibliography includes various methods that suggest respective formulas for a realistic estimation of actions on ties. The load that derives when applying these formulas under the most adverse conditions, gives values that justify sporadic appearance of cracks (in the order of 1-2%) but do not justify at all their systematic appearance at 60% of the ties. The most commonly utilized formulas are found in the German (Eisenmann et al, 1984) and in the French bibliography (Alias, 1984, Prud' home 1969, 1970, Prud' Homme A., Erieau J.,1976). These formulas, however, do not justify such an extensive appearance of cracks. Assuming the most adverse loading conditions only sporadic appearance of cracks (in the order of 1-2%) is justified when applying the aforementioned methodologies to calculate the actions on ties. The above facts generated the need of a more exhaustive investigation of the extensive appearance of cracks on ties, that would lead to the development of a new methodology for the calculation of the actions on the ties, which would be able to simulate and explain the extended cracking phenomena that have been observed in the Greek network. We have to mention that in the international bibliography there are repeated references of cracking of concrete ties (FIP 1987, FRA 1983, etc.)

The Greek railway network follows the international standards for permanent way and rolling stock. At the era, the conditions of the Greek Railway network included heavy non-suspended masses, limestone ballast – very often below the minimum standards of European networks with advanced technology – lack of grinding of the rail running surface, the maintenance method, great wheel flats, etc. The formula that was finally proposed (Giannakos 2004), was able to interpret the phenomenon of the systematic appearance of cracks that was observed in a high percentage of ties (Giannakos et al, 1994).

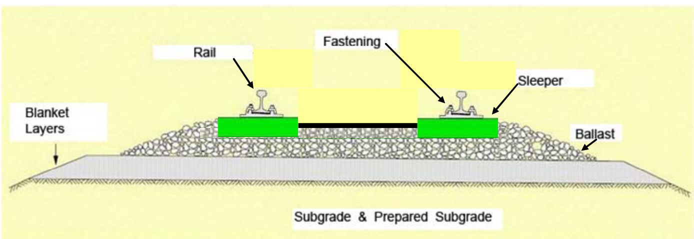

Fig. 1: Typical section of ballasted track with twin-block concrete ties

STRENGTH OF CONCRETE TIES – ACCEPTANCE CRITERIA

According to the French standards (Norme Francaise, 1989) and the SNCF standards (SNCF, 1980), since the sleepers U2 and U3 were produced in 1972-1978 and the tests of the research program were conducted in 1988 and 1989, the dynamic testing of ties was designed to correspond to three load regions as depicted in Fig. 2 (Norme Francaise NF F 51-101/Decembre, 1989; SNCF - VRE 2321 8–06 (B), 1980). These regions were determined -in the aforementioned French standards- by tests conducted under adverse seating conditions (Giannakos et al., 1988) and are described below (Relevant tests are included in EN 13230-2 “Railway applications - Track – Concrete sleepers and bearers - Part 2: Prestressed monoblock sleepers”, March 2003/ (DIN), page 13,).

1st region or region R1 (Pre-cracking stage): Appearance of the first dim cracks: this region is, in general, of little importance, because it varies a lot, according to the tensile strength of the concrete (reinforcement does not undertake any stresses at this point). The strength of the tie itself is only slightly affected by these cracks. This load region reaches 100 kN (~10 t) and corresponds to adverse seating conditions of the tie. In this region normal opeation loads are acting , that is the static loads with the increment of low frequency dynamic loads (the static load, the load from cant deficiency, the load from the suspended masses of the vehicles).

2nd region or R2 region (Post-cracking Service Load stage): Noticeable cracks appear, whose opening remains ≤ 0.05mm after unloading, and they disappear after unloading. These cracks do not obstruct track operation (that is, despite the cracks, the support conditions of the rail are ensured). This region begins between 125 and 130 kN for U2 and U3 twin-block concrete ties. Thus the load of this region for the laboratory test must be 125 kN ≥ R ≥130 kN for U2 and U3 ties. In this region exceptional dynamic loads are acting, which are, nevertheless, frequent on the track. These overloads are generated –mainly– from the non-suspended (or unsprung) masses (NSM), from the ordinary defects of the rail running table, such as bad welds or wheel burns etc, and also from the ordinary defects of the wheels. These loads refer to loads that are beyond the normal operation (service) loads of the “wheel-rail-tie-ballast-substructure” system (that is, exceptional overloads which, nevertheless, frequently appear on track).

3rd region or R3 region (Post-servicability Cracking stage): The cracks remain open after unloading with opening ≥ 0.5 mm. This stage, at its upper limit, precedes and practically characterizes the complete failure of the tie and is situated at 140 kN ≤ R3 ≤ 175 kN (U3 and U2 ties). In this region exceptional overloads appear, which are not frequently observed, such as: forgotten fastening clips on the rail running table, rail ruptures, gaps on the rail running table due to a shelling of a rail head section, wheel flats that exceed the acceptable tolerances, etc.

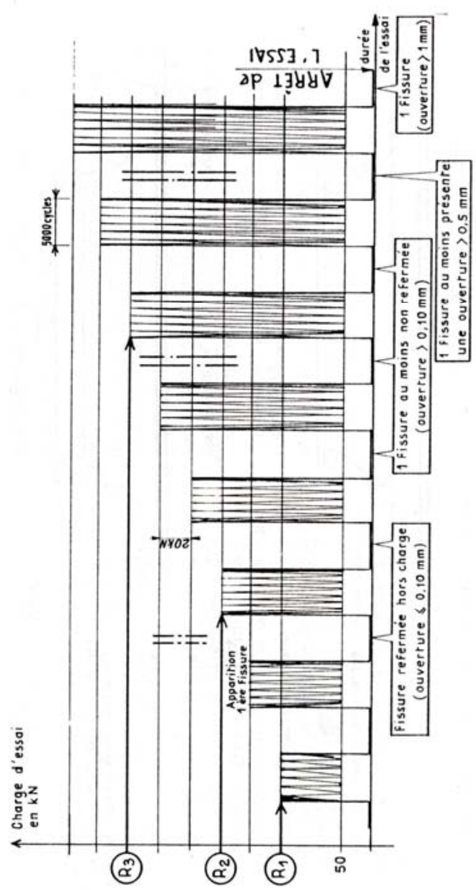

Fig. 2: Load Program for the acceptance test of sleepers (According to French regulations, Prud’homme et all. 1976)

ESTIMATION OF THE ACTIONS ON THE SLEEPERS

General

The theoretical approach for the most precise identification possible of its probable value, demands the analysis of the total load to individual component loads-actions, which, in general, can be analysed into static load, and dynamic load.

Static Load

The static load of a sleeper, in the classical sense, is the load undertaken by the tie when a vehicle axle at standstill is situated exactly above the location of the tie. At low frequencies, however, the load is essentially static. The static load is further analyzed into individual component loads.

Static Load due to Wheel Load. The most widely used theory (referred to as the Zimmermann theory or formula) examines the track as a continuous beam on elastic support whose behavior is governed by the following equation (Giannakos, 2004):

$$\frac{d^4 y}{dx^4} = -\frac{1}{E \cdot J} \cdot \frac{d^2 M}{dx^2} \qquad (1)$$

where y is the deflection, M is the moment that stresses the beam, J is the moment of inertia of the rail, and E is the modulus of elasticity of the rail.

From this formula it is concluded that the reaction of a tie is:

$$R_{sl} = \frac{Q_{wh}}{2\sqrt{2}} \cdot \sqrt[4]{\frac{\ell^3 \cdot \rho}{E \cdot J}} \qquad \Rightarrow \frac{R_{sl}}{Q_{wh}} = \overline{A} = \overline{A}_{stat} = \frac{1}{2\sqrt{2}} \cdot \sqrt[4]{\frac{\ell^3 \cdot \rho}{E \cdot J}} \qquad (2)$$

where Q_{wh} the wheel load, ℓ the distance between the ties, E and J the modulus of elasticity and the moment of inertia of the rail, R_{sl} the static reaction-load of the tie, ρ reaction coefficient of the tie which is defined as: ρ=R/y, and is a quasi coefficient of track elasticity (stiffness).

In reality, the track consists of a sequence of materials –in the vertical axis– (substructure, ballast, tie, elastic pad, rail), that are characterized by their individual coefficient ρ_i. Hence, for each material it is:

$$\rho_i = \frac{R}{y_i} \Rightarrow y_i = \frac{R}{\rho_i} \Rightarrow y_{total} = \sum_{i=1}^{\nu} y_i = \sum_{i=1}^{\nu} \frac{R}{\rho_i} = R \cdot \sum_{i=1}^{\nu} \frac{1}{\rho_i}$$
$$\Rightarrow \frac{1}{\rho_{total}} = \sum_{i=1}^{\nu} \frac{1}{\rho_i} \qquad (3)$$

where ν is the number of various layers of materials that exist under the rail –including rail– elastic pad, tie, ballast etc.

Load due to Cant Deficiency. This load is produced by the centrifugal acceleration exerted on the wheels of a vehicle that is running in a curve with cant deficiency. It is not, however, a dynamic load in the sense of the load referred to in the next paragraph. Therefore, it is often considered to be a semi-static load. The following equation (Giannakos et al 1988, 1990, 1994, Alias 1984):

$$Q_\alpha = \frac{2 \cdot \alpha \cdot h_{CG}}{e^2} \cdot Q_{wh} \qquad (4)$$

provides the accession of the vertical load of the wheel in the equation (2), at curves with cant deficiency. In the above equation α is the cant deficiency, h_{CG} the height of the centre of gravity of the vehicle from the rail head, e the track gauge, and Q_{wh} the wheel load. This semi-static load is also distributed with $\bar{A}_{stat}$.

Dynamic Load

Dynamic Load Calculation According to the German Bibliography. In the German bibliography, the total load (static and dynamic) acting on the track Q_{total} is equal to the static wheel load multiplied by a factor:

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

where: Q_{wh} is the static load of the wheel,
$\overline{s}$ = 0.1φ ÷ 0.3φ dependent on the condition of the track, that is:
$\overline{s}$ = 0.1φ for excellent track condition
$\overline{s}$ = 0.2φ for good track condition
$\overline{s}$ = 0.3φ for poor track condition

and φ is determined by the formulas proportionally to the speed:
for V < 60 km/h then φ = 1,
for 60<V<200 km/h then:

$$\varphi = 1 + \frac{V - 60}{140} \qquad (6)$$

where V the speed 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 of each tie (or, alternatively, the action on it) is calculated from the total load Q_{total} acting on the track. From german bibliography (Eisenmann J., 2004) the action (or reaction) on each tie can be derived (Where L is the "elastic length" of the track):

$$\frac{R_{max}}{Q_{total}} = \frac{Q\ell}{2L} = \frac{1}{2\sqrt{2}} \cdot \sqrt[4]{\frac{\ell^3 \cdot \rho}{E \cdot J}} = \overline{A}_{stat} \qquad (7)$$

If we apply the $\bar{A}_{stat}$, as the German bibliography refers, the derived values of the actions on the ties are very small and no cracking at all is expected (see Figure 4). In this paper and for the most adverse scenario the use of the dynamic coefficient of elasticity $\bar{A}_{dyn}$ instead of the static $\bar{A}_{stat}$ is proposed and considered contrary to the methodology described in the German bibliography. The following equations (Giannakos 2004) for $\bar{A}_{dyn}$ (see Figure 5):

$$\rho_{dyn} = h_{TR} = 2\sqrt{2} \cdot \sqrt[4]{E \cdot J \cdot \left(\frac{\rho}{\ell}\right)^3} \Rightarrow \bar{A}_{dyn} = \frac{1}{2\sqrt{2}} \cdot \sqrt[4]{\frac{\ell^3 \cdot \rho_{dyn}}{E \cdot J}} \quad (8)$$

<u>Dynamic Load Calculation According to the French Bibliography</u>. According to the French bibliography (Alias 1984, 1987, Prud' home 1969, 1970, Prud' Homme A., et al., 1976) the dynamic load consists of two components generated from: (a) the non-suspended (unsprung) masses, and (b) the suspended (sprung) masses.

<u>(A) Load caused by the non-suspended masses (NSM)</u>. The theoretical analysis leads to the following equation for the standard deviation of the load that is caused by the NSM (Alias 1984, Prud' home 1969 & 1970, see also Giannakos 2004):

$$\sigma(\Delta Q_{NSM}) = k_{\alpha} \cdot V \cdot \sqrt{m_{NSM} \cdot h} \quad [t] \quad (9)$$

where $\sigma(\Delta Q_{NSM})$ is the standard deviation of the dynamic component due to the non suspended masses that participates in the increase of the static load as described below, k'_{α} coefficient depending on the rail running table geometry, V the speed in km/hr, m_{NSM} in [t] the non suspended mass, and h the track stiffness in kN/mm.

<u>(B) Load Caused by the Suspended Masses</u>. The standard deviation of the load is given by the Eqn (10) (SNCF 1981, Prud' home 1969). Within the theoretical model of non-suspended (unsprung) masses (Fig. 3), and considering the model of the "single-floor" vehicle with one spring and one damper in parallel arrangement, the motion of the suspended (sprung) masses can be simulated.

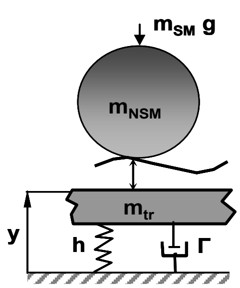

Fig. 3: Wheel on a rail as an infinite beam on elastic foundation

In this case excitation n will not represent the rail running table defects, but the motion of the wheel (analytical theoretical approach is cited in Alias 1984, Prud' homme 1970).

$$\sigma(\Delta Q_{SM}) = \frac{V - 40}{1000} \cdot N_L \cdot Q_{wh} \quad (10)$$

where N_L is the mean standard deviation of the longitudinal level condition of the track, equal to 0.7, in average, for the French network.

<u>(C) Total Dynamic Load</u>. For the calculation of the loads on the track two times the standard deviation must be taken into account (Alias 1984, p 206-207).

$$Q_{total\ max} = Q_{wh} \cdot \left(1 + \frac{Q_{\alpha}}{Q_{wh}} + 2 \cdot \frac{\sqrt{\left[\sigma^2\left(\Delta Q_{NSM}\right)\right] + \left[\sigma^2\left(\Delta Q_{SM}\right)\right]}}{Q_{wh}}\right) \quad (11a)$$

Besides the $\bar{A}_{stat}$ in the French bibliography there is no clear reference to the tie's reaction with the exception of a coefficient equal to ($1.35\bar{A}_{stat}$) in Prud'homme et al., 1976. If we apply the $1.35\bar{A}_{stat}$, as coefficient of reaction of the tie, for the distribution of the total load acting on track (static and dynamic), the derived values are very small and no cracking at all is expected (see Figure 4). We can conclude that to calculate the reaction R of the tie, $\bar{A}_{stat}$ for the static and semi-static components and no distribution for the dynamic component of the total load is used, even if the French bibliography presents only $\bar{A}_{stat}$.

$$Q_{total\ max} = Q_{wh} \cdot \left(\bar{A}_{stat} \cdot \left(1 + \frac{Q_{\alpha}}{Q_{wh}}\right) + 2 \cdot \frac{\sqrt{\left[\sigma^2\left(\Delta Q_{NSM}\right)\right] + \left[\sigma^2\left(\Delta Q_{SM}\right)\right]}}{Q_{wh}}\right) \quad (11b)$$

<u>Dynamic Load Calculation with the Use of Formulas Derived for the Conditions of the Greek Network</u>.

<u>(A) Load caused by the non-suspended masses (NSM)</u>. Since OSE does not have measurement data available from its own network to calculate $\sigma(\Delta R_{NSM})$, it was jointly proposed by a scientific team of OSE, National Technical University of Athens and SNCF, to apply the comparative equation, and set as reference base a measured track of SNCF with: 200km/h, (m_{tr}+ m_{NSM}) = 1.7804t, and $\rho_{dyn} = h_{TR}$ = 75kN/mm (Giannakos et al, 1988, Giannakos 2004, Giannakos et al., 2007). This yields the following equation:

$$\sigma\left(\Delta Q_{NSM}\right) = k'_{\alpha} \cdot \frac{V}{200} \cdot \sqrt{\frac{m}{1.7804}} \cdot \sqrt{\frac{h}{75}} \quad [t] \quad (12)$$

where m in [t] is the NSM of vehicle and track, V is the speed in km/hr ,and h is the dynamic stiffness of the track in kN/mm for the track under examination given by equation (13) below (Giannakos 2004).

It is easy to prove that: k'_α fluctuates from 0.9 for newly ground rail to 3.6 for a used rail just before re-grinding (Giannakos et al., 2007).

Finally, ρ_{dyn} or h_{tr} is calculated (Jenkins et al 1974) through the following equation:

$$h_{TR} = \frac{2 \cdot \rho}{f \cdot \ell} \qquad where \qquad f = \left[\frac{\rho}{4 \cdot E \cdot J \cdot \ell}\right]^{0.25} \tag{13}$$

$$and \quad h_{TR} = 2\sqrt{2} \cdot \sqrt[4]{E \cdot J \cdot \frac{\rho_{total}}{\ell}}$$

which is the same as Eqn (8).

(B) Load Caused by the Suspended Masses.

For the suspended masses equation (10) is used. In the same bibliography (Cooperation OSE/SNCF 1988, 1989, Giannakos et al. 1988, 1994), N_L is given as the standard deviation of the longitudinal level defects along the track. According to the experts assessment of the French and Greek railways during the research program of 1988-1989, for the Greek network N_L has to be equal to 1mm.

(C) Proposed Distribution of Loads and Value for the Acting Load on Ties. In high frequencies, the response of the superstructure is negligible due to its low eigenfrequency, therefore, for safety reasons, it is assumed that dynamic loads (semi-statics due to cant deficiency are also included) are not distributed to the adjacent ties, in contrast to static loads. It is recommended that the service load should be equal to the static load increased by 3 times the standard deviation of the dynamic load (P = 99.7%):

$$R_{serv} = \bar{A}_{dyn} \cdot \left(Q_{wh} + Q_\alpha\right) + (3 \cdot \sqrt{\left[\sigma^2\left(\Delta Q_{NSM}\right)\right]^2 + \left[\sigma^2\left(\Delta Q_{SM}\right)\right]^2}) \tag{14}$$

where $\bar{A}_{dyn}$ is calculated by equation (8) where instead of ρ_{total} the dynamic stiffness h (or ρ_{dyn}) of the track is used, given by the Eqn (8) or Eqn (13).

Experimental research and measurements have also been conducted in the laboratories of the Reinforced Concrete Department of the NTUA (Tassios et al. 1989, Abakoumkin et al. 1992), the Geotechnical Engineering Department of the Aristotle University of Thessaloniki (Tsotsos 1989), the French Railways (SNCF), the Hellenic Ministry for the Environment, Physical Planning and Public Works/Central Public Works Laboratory, the tie factory of OSE, but also on track in the Athens-Thessaloniki axis (Riessberger 1992). Based on: (a) the situation observed and recorded by the research conducted on the Greek railway network, (b) the available data from measurements at foreign networks, and (c) published research data, the authors concluded on definite fluctuations in the values, of the individual parameters that approximate the Greek reality (Giannakos 2004).

NUMERICAL APPLICATION ON A U3 TWIN-BLOCK TIE OF THE GREEK NETWORK

The conditions of the Greek network between the 1980s and the beginning of 1990s, consisted of very compacted, polluted ballast bed and stiff support (ρ_i = 380 kN/mm) and substructure classified according to the fluctuation of coefficient ρ_i for the seating of the track from :

- ρ_i = 40 kN/mm for pebbly substructure to the most adverse conditions of
- ρ_i = 100 kN/mm, which corresponds to frozen ballast bed and substructure or approximately the rigidity of NBS1 of the DB (107 kN/mm),
- ρ_i = 250 kN/mm for stiff (rigid) subgrade at the bottom of a tunnel or on a concrete bridge with very small height of ballast-bed.

The rest of the parameters of the track that influence the state of actions on ties and possibly led to the appearance of cracks in Greece are: nominal maximum speed V = 120 km/h which in practice was exceeded permanently for many years up to 140 km/h (this value is used), NSM = 2.55 t (three-axle bogies in Romanian diesel-locomotives of Electropoutere type, in Greek network), UIC54 rail, 4.5mm elastic pad, and 105 mm cant deficiency.

The calculations according to the proposed method are performed with the program –included in the book "Actions on the Railway Track" (author Giannakos K., Papazissis publications, Athens, 2004). The results are depicted in Figs 4, 5 and 6.

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.

So the method –included in the regulations- for calculating the pad stiffness from two discrete values (i.e. 18 and 70 kN) of load is not describing the real situation, where an equilibrium among the various "springs" that comprise the system of the track takes place. The trial-and-error method must be utilized in order to more accurately estimate the stiffness of the pad in each case. In this paper the stiffness of the pad is calculated with the trial-and-error method and then the acting forces-loads on the twin-block ties with the RN fastenings are calculated. The same procedure is followed for the Skl-14 tension clamp with the "soft" Zw700 pad as well as for the

W24 fastening with Zw700 WIC for heavy haul rail transport cited below. The results of the calculations are compared with the real situation of the track in the Greek network, where the twin-block concrete ties presented extended cracking, having exceeded the R2 and R3 limits. This comparison is done for the W14-Fastening which has not presented yet any cracking at all.

In Fig. 4 the results of the calculations are presented, for the RN fastening and the W14 fastening, according to the different methods described in this paper. The distribution of the total load is done using the coefficient of sleeper's reaction according to the french and german bibliographies (Prud'homme et al., 1976, Eisenmann 2004). It is clear that while the equation from the german bibliography is not affected at all by the situation of the rail running table, the "equilibrium" which is taken into account from the trial-and-error method in the proposed equations gives almost the same results. The results of equation (14), proposed in this paper, are presented in relation to the equations (5) in combination with $\overline{A}_{stat}$ (german see Eisenman 2004) and equation (11a) in combination with $1.35*\overline{A}_{stat}$ (French see Alias 1984, Prud' Homme et al 1976), as a comparison of actions-loads that are calculated in each case as service loads for the dimensioning of the cross section of the semi-tie. The results of the formulas and calculation method given in german bibliography –for high-speeds- are presented for the case of extreme possible values α= 0.3 and t = 3. It is obvious that in conditions where ρ_i = 100 kN/mm, this equation yields results that are clearly under the limit of 125 kN therefore even no sporadic cracking is expected. It is characteristic that German bibliography (e.g. Fastenrath 1981) presents a detailed description of the formulas and calculation method. It is assumed that there is a distribution of the total load, Q_{total}, to the adjacent ties. Nevertheless there is no mention of the load that is undertaken by each tie, with the exception of Eisenmann J., 2004.

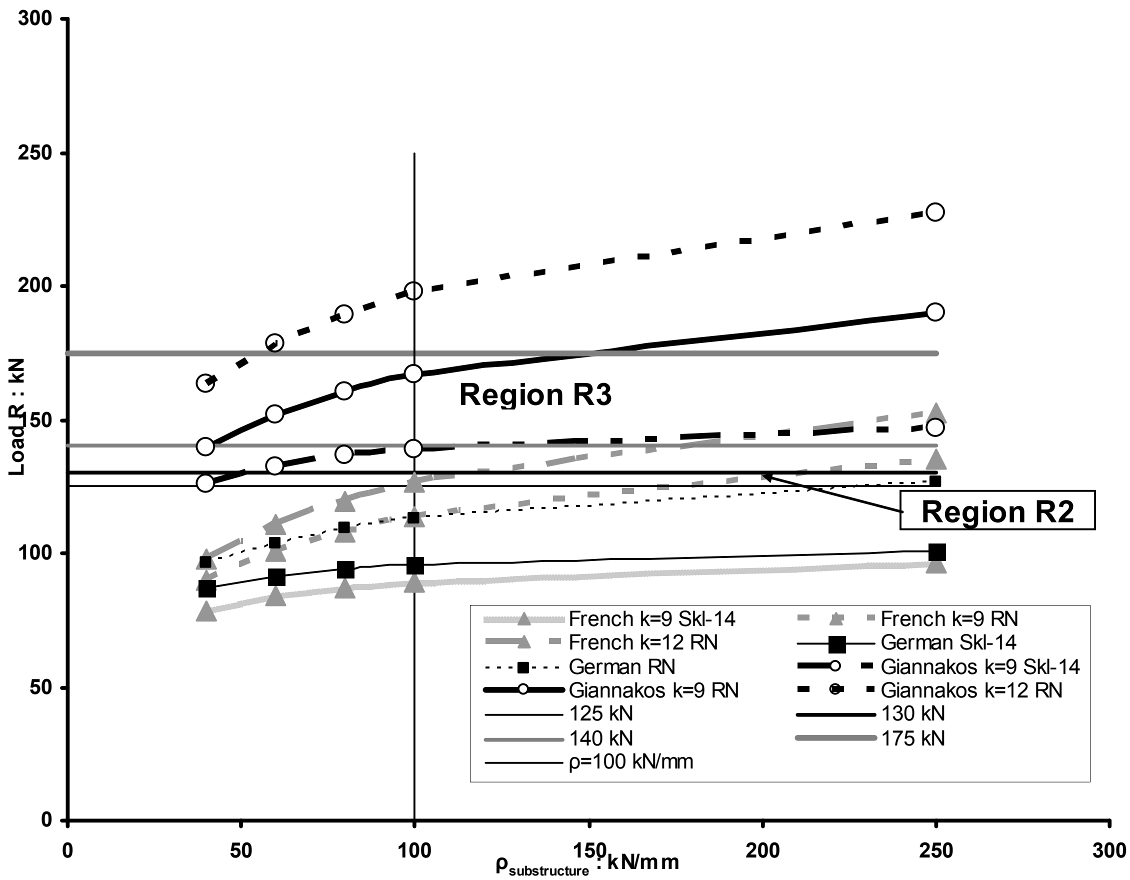

Fig. 4: Actions on ties according to: (a) French bibliography with distribution of load using the reaction coefficient $1.35\overline{A}_{stat}$, (b) German bibliography with distribution of load using the reaction coefficient $\overline{A}_{stat}$, and (c) Giannakos

In Fig. 5 (see also Giannakos et al., 2007) the results of the aforementioned calculations are also presented, for the RN fastening and the W14 fastening, according to the different methods described in this paper, but with a change for the equations derived from the german and french bibliographies: the distribution of the total load is done using a more adverse coefficient of sleeper's reaction according to the proposals of the greek research team of the era, in which the author participated. Even in this case, the equation from the German bibliography is not affected at all by the situation of the rail running table, the "equilibrium" which is taken into account from the trial-and-error method in the proposed equations gives almost the same results. The results of equation (14), proposed in this paper, are presented in relation (a) to the combined equations (5) -derived from the German bibliography- and (8) and (b) equation (11b) for the sleeper's reaction derived from the (11a) cited in the french bibliography, as a comparison of actions-loads that are calculated in each case as service loads for the dimensioning of the cross section of the semi-tie. The results of the formulas and calculation method given in German bibliography –for high-speeds- are presented for the case of extreme possible values α= 0.3 and t = 3 and for the reaction on the tie by using $\bar{A}_{dyn}$. It is obvious that in conditions where ρ_i = 100 kN/mm, this equation yields results that are around the limit of 125 kN therefore sporadic cracking is expected (in the order of 1 to 2%).

The results of the equation (11b) and calculation method provided in the French bibliography (Alias 1984) are presented with the assumption of load distribution according to Eqn (11b) and for two values of k_1: 9 and 12. The latter is the extreme acceptable value –after the appearance of which, grinding of rail running table must ensue– and this situation has never occurred systematically in the Greek network. This formula and calculation method, in conditions of ρ_i = 100 kN/mm and extreme values of the rail running table k_1= 12 gives results on the limit of 140kN, therefore sporadic cracking ≤0.05 mm is expected in the order of 1 to 2%. For a mean condition of rail running table, k_1= 9, the equation gives results that do not indicate any cracking whatsoever, or at least sporadic dim cracks.

CONCRETE TIE DESIGN LOAD

Before the development of high-resilience fastenings

As already mentioned, before the development of high-resilience fastenings, twin-block ties and RN fastenings were in use in the Greek network. The service load (deriving from equation 14) is given for values of k=9 and 12, maximum speed V=140km/h, cant deficiency 105 mm, and NSM=2.55 t.

These values are characteristic of the most adverse conditions in the Greek railway network at the era for a UIC54 rail and 4.5 mm pad of RN (fastening of French technology) with old type of very rigid and in some cases very soft substructure as measurement on site have proven (Tsotsos 1989, Riessberger 1989). The calculation is done graphically from the load-deflection curve of the elastic pad of the fastening. The accuracy of the graphic method is satisfactory in comparison to the magnitude of the forces acting on track.

From Fig. 4 and 5 the value of the service load of the tie can be obtained for design purposes, applicable for fastenings of elasticity and conditions identical to those of the Greek network of the 1980s.

These values are derived for substructure cases where there are concrete bridges or rocky subgrade (e.g. the bottom of tunnels with very small or practically non-existent height of the ballast under the tie). These cases nevertheless are "spot cases" for a railway network. If measures are taken to increase the total elasticity of the line at these spots, then respectively a service load R_{serv} < 200 kN can be derived (similarly for ties equipped with a fastening of this elasticity and conditions identical to those of the Greek network in the 1980s.). Normally, a representative value of ρ_{substr} illustrating the situation in the Greek network at the era was 100 kN/mm at maximum.

Influence of High-Resilience Fastenings

Research around the world has led to the production of a new "generation" of very resilient fastenings (e.g. German W14), that reduce the load on ties. In Fig. 4 and 5 the values of the service load are presented for W14 fastening with k=9 and 12, maximum speed V=140km/h, cant deficiency 105 mm, NSM=2.55 t, and UIC54 rail. The comparison of the service load between the german W14 fastenings and the stiffer ones (e.g. french fastenings RN) shows that the load on the ties is significantly reduced in the case where more resilient fastenings are utilized. Moreover it is clear that high elastic rail pads reduce not only the load per tie, they even bring more advantages, as:

-Higher passenger comfort
-Protection of the ballast by damping the vibrations and bumps on the ballast and reducing the load per rail seat
-Protection of the rolling material

Conclusions from the Greek Experience

In Greece, today, the situation has been improved with the adoption of «softer» fastenings (W14), the use of ballast of greater hardness DRi ≥14, new modern rolling stock with lower value of non-suspended (unsprung) masses, the adoption of grinding of the rail running table, construction of substructure of higher quality and specifications (Proctor 100% or modified Proctor 105%) and –perhaps in the future– the adoption of shorter distances among the ties (e.g. 55cm) (Tsoukantas S., Giannakos K., et al 1999) to improve of the poor geotechnical conditions of the subgrade. Fig. 4 and 5 present the load that twin-block (or monoblock) ties would undertake with W14 fastening instead of RN fastening, for the same conditions as mentioned above. It is concluded that no crack at all would be observed – even on rocky substructure in tunnels or concrete bridges, with the exception of sporadic ones in this case (max. load = 147.12 kN for ρ_{substr}=250 kN/mm and max. load = 138.86 kN for ρ_{substr}=100 kN/mm).

In practice, after almost ten years of track in operation with monoblock B70 type ties with W14 fastening, in Greece with operational maximum speed 160 km/h now instead of 140 km/h at the past, these conclusions are verified by the fact that there is no appearance of cracks in the Greek railway network and their excellent behaviour in the track. The adoption of high-resilience fastenings – such as the W14 that was adopted in Greece- is of crucial importance, in combination with the construction of higher quality substructures. This gives a very smooth curve of actions on the ties in relation to the ρ_{total} of the track and the actions on the ties are clearly under the limits. Finally, measures such as the treatment of rail running table, the decrease of the non-suspended masses of the vehicles, and the restriction in the use of limestone ballast have to be undertaken.

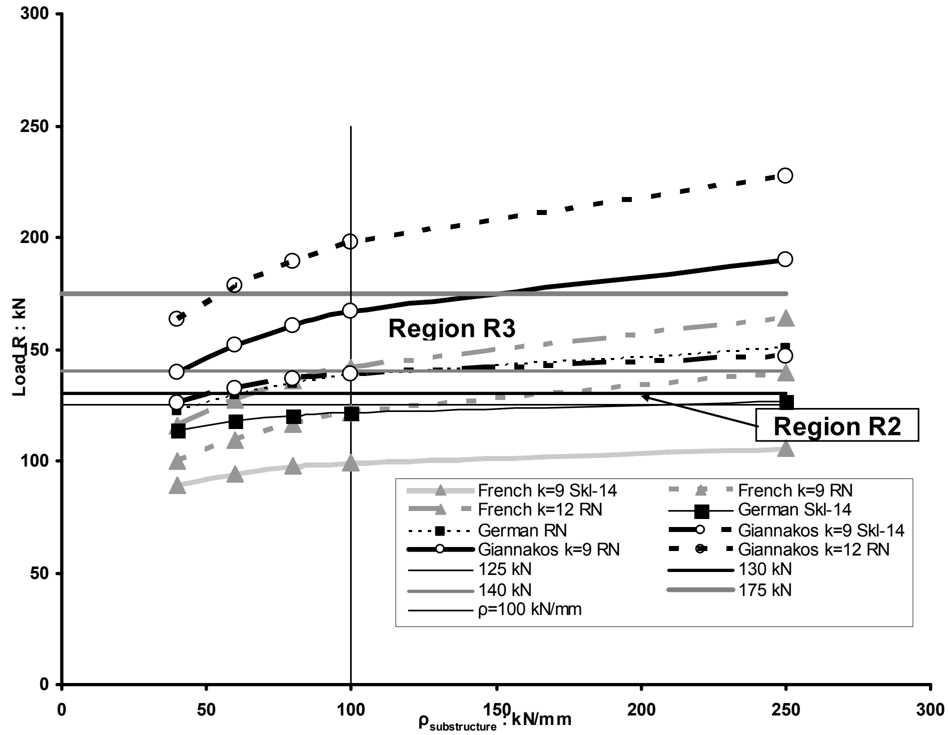

Fig. 5: Actions on ties according to: (a) French bibliography with distribution of load according to the author's proposal, (b) German bibliography with distribution of load according to the author's proposal, and (c) Giannakos

Application in the Heavy-haul Railways in USA

In the United States of America the heavy-haul freight railway transportation has different characteristics: wheel load 17.69 t (35.38 t per axle), maximum speed 60 mph that is 96.6 km/h or approximately 100 km/h, distance between two consecutive ties 24 inches or 60.96 cm. Ahlbek D. R., et al. (1978) and Hay W., (1982) propose values in the order of 4450 lbs per wheel for the unsprung (non-suspended) masses. The author was unable to receive up to now any updated data either from the industry or the Railroads in the States. In this paper this value is used for the unsprung masses.

The adoption of high-resilience fastenings – such as the W24 that can be adopted for switches (and plain track also) with the elastic pad Zw 700 WIC, will help decisively in the reduction of stresses and consequently to the prolongation of life-cycle and of the maintenance costs of the track.

Applying the proposed method in this paper, with the aforementioned trial-and-error procedure for the load-deflection curve of the pad, we can find the values of the actions on the ties as in Fig. 6, for the above parameters and 69.25 kg/m rail 140RE type (AREMA 2005, FIP 1987), concrete sleeper 2.59 m ~ 2.60 m (363 kg), and rail running table in two conditions k=9 and 12. For comparison reasons the values for the RN and W14 fastenings for the Greek conditions are depicted also.

CONCLUSIONS

The railway track superstructure undertakes the forces that develop during train movement and distributes them towards its seating (subgrade). The ties constitute a substantial element of the superstructure, especially as far as load distribution is concerned, while at the same time they ensure the stability of the geometrical distance between the rails.

In this paper a method has been presented for the calculation of the loads acting on concrete ties and the track superstructure and substructure. A parametric investigation for various geotechnical conditions (poor to excellent) of the substructure has been conducted. The results of the proposed method are verified in practice. The method was initially developed because of the weakness of the existing commonly used theories to explain the systematic appearance of cracks on ties laid on the Greek network. The proposed method is applicable to all Railway networks that are compliant to the interoperability specifications as the members of the EU.

The main differences between the proposed method and the existing ones are that: (a) the static component of the total load acting on a tie is derived after a distribution through the $\bar{A}_{dyn}$ (instead of $\bar{A}_{stat}$), (b) the dynamic component of the total load acting on a tie is increased by three times the standard deviation (instead of two in the French method), (c) the dynamic component of the total load is not distributed to the adjacent ties but it is considered acting directly on one tie and (d) for the estimation of the service load, the tie-pad stiffness is calculated through a trial-and-error procedure that ensures equilibrium among the numerous springs-components of the track system for discrete geotechnical conditions of the substructure.

Finally, modern, high-resilience fastenings (e.g. Vossloh W14 that has been laid in the Greek railway network since 2000) significantly reduce the actions on the concrete ties and track substructure, and therefore their use must be obligatory in the modern railway tracks since they eliminate the problems created by the poor geotechnical conditions of the track substructure.

The application of the proposed method for the heavy-haul rail transportation in the USA with axle-load 35.38 t, in a track equipped with fastenings W24 results in actions on the ties smaller than or approximately equal to the values in the Greek network with axle-load 22.5 t and RN fastening. Finally the W24 fastening gives very good results for heavy-haul rail transport. It is worth-mentioning that for worse running rail table (e.g. k=12) the attenuation of the impact loads resulting from W24 is much greater. This reduction of the actions on the track's superstructure reduces significantly the annual maintenance cost according to the AASHO road test:

Decrease in track geometry quality =
(increase in stress on the ballast bed)^m

where m varies between 3 and 4.

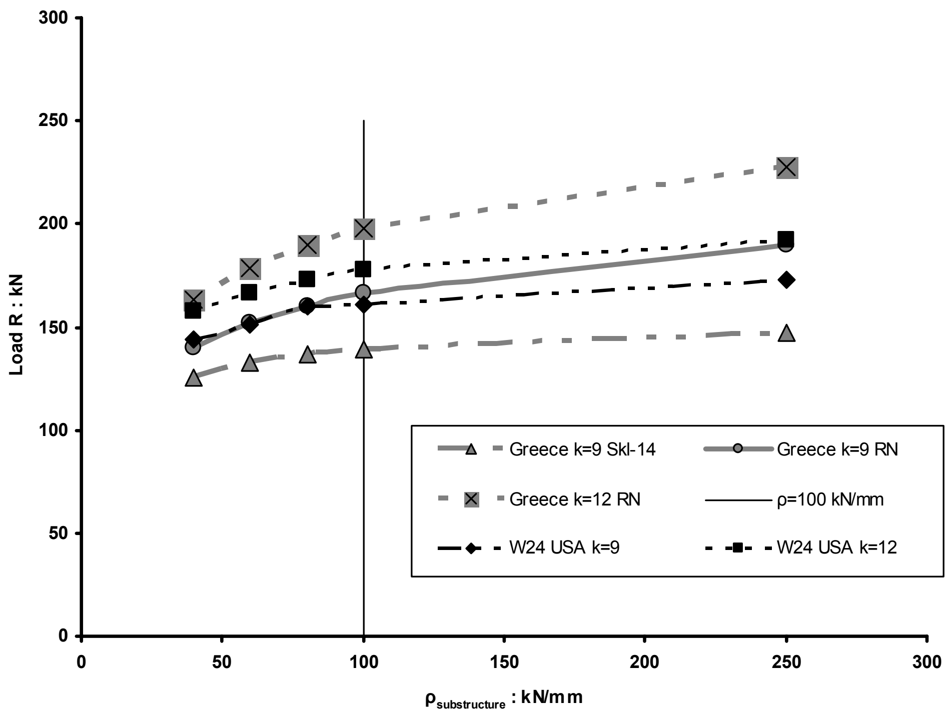

Fig. 6: Actions on ties in the USA (heavy-haul) in a track with 140RE rail – 69.25 kg/m, concrete ties, W24 fastening with Zw 700 WIC elastic pad, in comparison with the actions on ties in the Greek network equipped with UIC 54 rails, concrete ties and fastenings RN and W14..

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