Design Loads on Railway Substructure - Comparative parametric Investigation on the Influence of Fastening Stiffness (European and Japanese)

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DOI: 10.13140/RG.2.1.1749.1924

2nd International Conference on Transportation Geotechnics - IS-Hokkaido, Japan, 2012

ABSTRACT: The existing international bibliography -German, French, AREMA methods- includes various methods for a realistic estimation of loads on track superstructure and the reactions/actions on the sleepers. The magnitude of loads derived from these formulas could not justify the systematic appearance of cracks at 60% of the sleepers in the Greek network. This generated the need of a more exhaustive investigation of the extensive appearance of cracks on sleepers, that would lead to the development of a new methodology for the calculation of the actions on the sleepers, which would be able to simulate and explain the phenomena that have been observed in the Greek network. In this paper a comparative parametric investigation is presented for the influence of fastening stiffness on the estimation on Design Loads on Railway Superstructure and Substructure. Japanese and European data are compared.

1 INTRODUCTION

The railway track superstructure is the equivalent of the road pavement and as in the case of pavements there are flexible and rigid track superstructures. The main difference between the two types of infrastructure is that the loads in railways are applied in two discrete locations along the rails, whereas in road pavements the location of the load application is random. The track superstructure is a multilayered system that consists of the rails, ties with their fastenings, ballast -the equivalent of a flexible pavement- and the blanket layer consisting of compacted sand and gravel, which further distributes the loads and protects the substructure from the penetration of crashed ballast particles, mud ascent and pumping. The stress on the substructure plays a key role in the design and maintenance of High Speed railway tracks and its magnitude mainly depends on the track stiffness coefficient (Giannakos, 2011). However, there is a lack of data in the international literature correlating the magnitude of stress on the track substructure and the track stiffness coeffient of High Speed lines under operation. The research performed for the Greek Railway network for the cracks observed on concrete sleepers in a percentage higher than 60% of the total number laid on track (Giannakos, 2008, Giannakos & Loizos, 2009) addressed this issue. Its findings, highlighting the interaction between superstructure and substructure of a railway track, are presented in this paper. A method for the calculation of loads and stresses on a railway track was developed as a result of this research (Giannakos, 2004). This method together with three methods found in the international literature are used to calculate the stresses on the track substructure and the results are compared and discussed for the fastenings in the Greek network and a comparison is presented for the Japanese fastenings (Giannakos, 2009-2010).

2 ACTIONS ON RAILWAY TRACK

It must be noted here that in all four calculation methods the total static stiffness coefficient of the track ρ_{total} (spring constant) is of decisive importance for the calculation of the action/reaction on each tie. In general:

$$\frac{1}{\rho_{total}} = \sum_{i=1}^{\nu} \frac{1}{\rho_i} \qquad (1)$$

where i are the layers that constitute the multilayered structure “Track”, and ρ_{total} the total static stiffness coefficient of track, which must be calculated for each case.

2.1. Method cited in the French literature

Given by (Alias,1984, Prud’homme & Erieau, 1976):

$$R_{total} = \left(Q_{wheel} + Q_a + 2\sqrt{\left[\sigma^2(\Delta Q_{NSM})\right] + \left[\sigma^2(\Delta Q_{SM})\right]}\right) \cdot \bar{A}_{stat} \cdot 1{,}35 \qquad (2)$$

which covers a probability of occurrence P=95.5%, where: Q_{wheel} = the static load of the wheel, Q_α = load due to cant (superelevation) deficiency, $\sigma(\Delta Q_{NSM})$ = standard deviation of the Non-Suspended Masses of vehicle, $\sigma(\Delta Q_{SM})$ = standard deviation of the Suspended Masses of vehicle, $\bar{A}_{stat}$ = reaction coefficient of the tie which is equal to:

$$\bar{A}_{stat} = \frac{1}{2\sqrt{2}} \sqrt[4]{\frac{\rho_{total} \cdot \ell^3}{E \cdot J}} \qquad (3),$$

and ρ_{total} = coefficient of total static stiffness (elasticity) of track, ℓ = distance between sleepers, E, J = Modulus of Elasticity and Moment of Inertia of the rail

2.2. Method cited in the German literature

According to Fastenrath, 1981, Eisenmann, 2004:

$$R_{\max} = Q_{wheel}\left(1+0.9\cdot\left(1+\frac{V-60}{140}\right)\right)\cdot\bar{A}_{stat} \qquad (4)$$

which covers a probability of occurrence P=99.7%, where: $\bar{A}_{stat}$ is given by equation (3), Q_{wheel} is the static load of the wheel, V the maximum speed.

2.3. Method cited in the American literature

According to AREMA 2005 (Hay, 1982, Selig & Waters, 2000, see also Giannakos, 2010a):

$$R_{\max} = p_{\max}\cdot\ell = \bar{A}_{stat}\cdot\left(1+\frac{D_{33}\cdot V}{D_{wheel}\cdot 100}\right)\cdot Q_{wheel} \qquad (5)$$

where D_{33} is the diameter of a wheel of 33 inches, D_{wheel} the wheel diameter of the vehicle examined in inches, p_{max} the maximum pressure per unit length of the track under the sleeper, and $\bar{A}_{stat}$ is the same as in equation (3).

2.4 Giannakos (2004) method

According to this method which was derived as a result of the research in the Greek railway network (Giannakos 2004, see also Giannakos& Loizos 2009):

$$R_{service} = \bar{A}_{dynam}\cdot\left(Q_{wheel}+Q_{\alpha}\right)+\left(\mu\cdot\sqrt{\left[\sigma\left(\Delta Q_{NSM}\right)\right]^{2}+\left[\sigma\left(\Delta Q_{SM}\right)\right]^{2}}\right) \qquad (6)$$

where: μ coefficient covering the probability of occurrence (μ=2,3,5 for P=95.5 %, 99.7%, 99.9%), $\sigma(\Delta Q_{NSM})$ is the standard deviation of the dynamic load due to non-suspended masses m_{NSM} of each axle, $\sigma(\Delta Q_{SM})$ is the standard deviation of the dynamic load due to suspended masses m_{SM},

$$\sigma\left(\Delta Q_{NSM}\right) = k_{1rail}\cdot V\cdot\sqrt{m_{NSM}\cdot h_{TRACK}} \qquad (7)$$

$$\sigma\left(\Delta Q_{SM}\right) = \frac{V-40}{1000}\cdot N_{L}\cdot Q_{wheel} \qquad (8)$$

k_{1rail} coefficient of the condition of the running rail table, fluctuating (for lines with $V_{max}\geq 140$km/h) between 0.00389 - 0.00584 for ground rail running table and 0.00779 - 0.01558 for non-ground rail running table, N_L coefficient normally equal to 1 - 1.2 and:

$$\bar{A}_{dynam} = \frac{1}{2\sqrt{2}}\cdot\sqrt[4]{\frac{\ell^{3}\cdot h_{TR}}{E\cdot J}} \qquad (9)$$

$$h_{TR} = \rho_{dynam} = 2\sqrt{2}\cdot\sqrt[4]{E\cdot J\cdot\left(\frac{\rho_{total}}{\ell}\right)^{3}} \qquad (10)$$

The calculation of the track mass participating in the motion of the Non-Suspended Masses is cited in Giannakos (2010c).

2.5. Comparison of theoretical calculations

In Greece between 1972 and 1999, twin-block concrete sleepers of French technology were exclusively used, with RN fastenings, for tracks designed for V_{max}=200km/h and temporary operational speed V_{oper}=120-140km/h. Extended cracking was observed at a percentage of more than 60 % of the sleepers laid on track. The methods cited in the international literature at that time did not provide any satisfactory justification for the appearance of the cracks, resulting in much lower values of actions on ties than the cracking threshold, thus predicting no cracking at all. After an extensive research that included collaboration among various universities and railway organizations in Europe, the Giannakos (2004) method was developed whose results successfully predicted the extended cracking of the U2/U3 ties (Giannakos, 2004, Giannakos & Loizos, 2009), calculating actions over the cracking threshold and in some cases over the failure threshold (Figure 1).

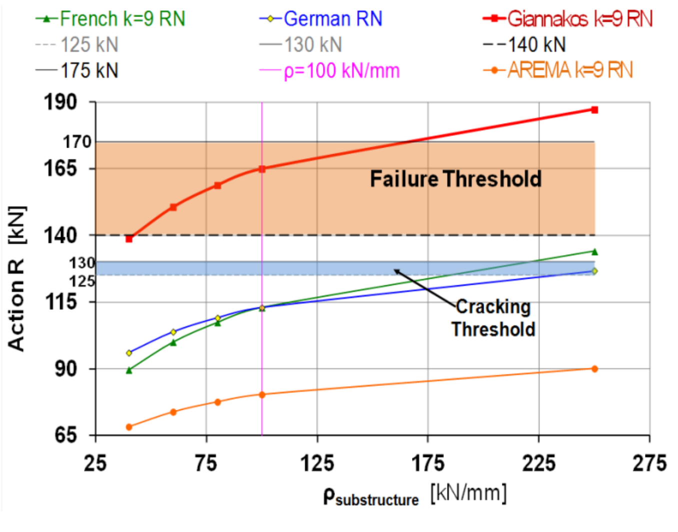

Figure 1: Comparison of the results of the 4 methods (German, French, AREMA, Giannakos) for U2/U3 sleepers with RN fastenings and 4,5 mm pad.

This method was derived from theoretical analyses, measurements from laboratory tests performed in Greece, Austria, France, Belgium and other European countries and observations from real on-track experience. The results of the method were also presented for lines with Heavy Haul traffic that are typical in the United States (Giannakos, 2011).

3 STIFFNESS COEFFICIENTS OF THE ELASTIC PAD - CALCULATION OF THE ACTIONS ON TRACK PANEL ACTIONS ON RAILWAY TRACK

The pad's stiffness plays a key role in the total response of the track. The pad's stiffness coefficient, ρ_{pad}, is calculated through the trial-and-error method from the load-deflection curve of the pad, provided by the producer. For the calculations the following combinations are considered based on the experience from the Greek railway network: UIC60 rail (ρ_{rail}=75000 kN/mm), 2,60 m concrete sleeper B70 type ($\rho_{sleeper}$=13500 kN/mm), distance between sleepers 60 cm, ballast 2 years in operation ($\rho_{ballast}$=380 kN/mm), subgrade with stiffness $\rho_{subgrade}$ fluctuating from 40 kN/mm in the case of pebbly subgrade to very rigid of 250 kN/mm in the case of rocky tunnel bottom or concrete bridge with insufficient ballast depth and fastening W14 with pad Zw700 Saargummi with stiffness ρ_{pad} calculated through the trial-and-error method, wheel load 11,25 t, maximum speed 250 km/h, Non-Suspended Masses 1,5 t, height of the vehicle's centre of gravity from the rail running table 1,5m, wheel's diameter 1 m (39,37 inch), maximum cant deficiency 160 mm, and average condition of an un-ground rail for the rail running table with coefficient ka'=0,0116873.

The actions have been calculated also for the verification of the model (Figure 1) for the case of the twin-block concrete sleepers that presented an extended cracking (in a percentage higher than 60%) of the total number of sleepers laid on track with the following data: UIC54 rail (ρ_{rail}=75000 kN/mm), twin-block concrete sleeper U2/U3 type ($\rho_{sleeper}$=13500 kN/mm), distance between sleepers 60 cm, ballast 2 years in operation ($\rho_{ballast}$=380 kN/mm), subgrade with stiffness $\rho_{subgrade}$ fluctuating from 40 kN/mm in the case of pebbly subgrade to very rigid of 250 kN/mm in the case of rocky tunnel bottom or concrete bridge with insufficient ballast depth and fastening RN with pad 4,5 mm with stiffness ρ_{pad} calculated through the trial-and-error method, wheel load 10,40 t, maximum speed 140 km/h, Non-Suspended Masses 2,54 t, height of the vehicle's centre of gravity from the rail running table 1,5m, maximum cant deficiency 105 mm, and average condition of an un-ground rail for the rail running table with coefficient ka'=0,0116873.

For comparison, the same conditions were applied for the first case in combination with the Japanese fastenings (as presented in Figure 2) with the relevant second kind pad of stiffness 60 MN/m, with actual stiffness ρ_{pad} in each case calculated through the trial-and-error method, and: UIC60 rail (ρ_{rail}=75000 kN/mm), 2,60 m concrete sleeper B70 type ($\rho_{sleeper}$=13500 kN/mm), distance between sleepers 60 cm, ballast 2 years in operation ($\rho_{ballast}$=380 kN/mm), subgrade with stiffness $\rho_{subgrade}$ fluctuating from 40 kN/mm in the case of pebbly subgrade to very rigid of 250 kN/mm in the case of rocky tunnel bottom or concrete bridge with insufficient ballast depth, wheel load 11,25 t, maximum speed 250 km/h, Non-Suspended Masses 1,5 t, height of the vehicle's centre of gravity from the rail running table 1,5m, wheel's diameter 1 m (39,37 inch), maximum cant deficiency 160 mm, and average condition of an un-ground rail for the rail running table with coefficient ka'=0,0116873.

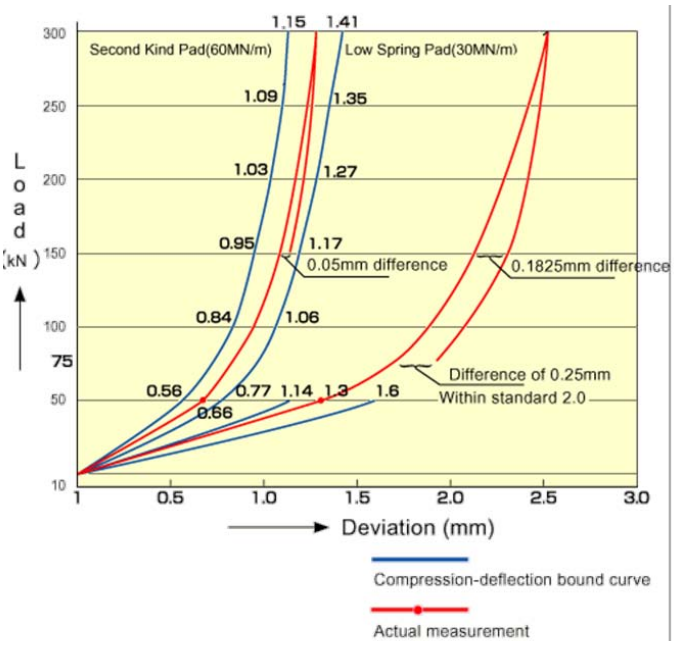

Figure 2: Load-Deflection curves of Japanese fastening pads.

The results for the Actions/Reactions on each support point (sleeper) of the track panel in the case of 2,60 m long sleeper B70 type of prestressed concrete and W14 fastening with pad Zw700 Saargummi, are depicted in Figure 3, as derived from the four methods described above: German, French, AREMA and Giannakos (2004).

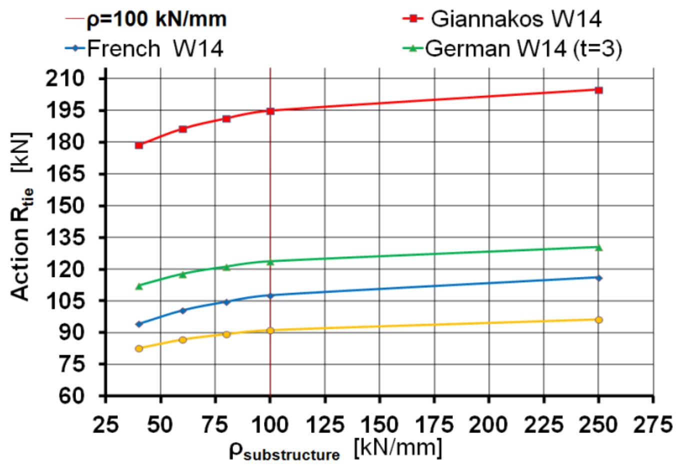

Figure 3: Actions on the track panel calculated by the four methods for fastening W14 and pad Zw700 Saargummi.

The results for the Actions/Reactions on each support point (sleeper) of the track panel in the case of 2,60 m long sleeper B70 type of prestressed concrete and Japanese fastening with pad of second kind (60 MN/m) of the Figure 2, are depicted in Figure 4, as derived from the four methods described above: German, French, AREMA and Giannakos (2004). Since there is no reference available, in this paper a toe-load of 9 kN per fastening (18 kN per rail) derived from the correct tightening of the "fastening's clip" is considered.

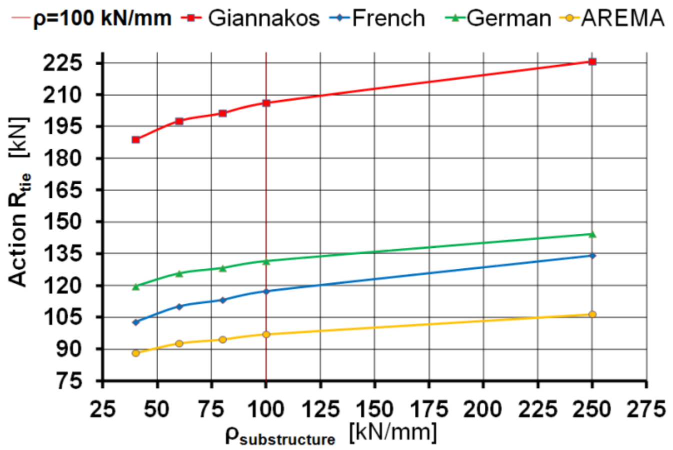

Figure 4: Actions on the track panel calculated by the four methods for Japanese fastening with pad of second kind (60 MN/m).

4 STRESSES ON BALLAST-BED, SUBGRADE AND DESIGN CRITERIA

The average stress $\bar{p}_{ballast}$ under the tie seating surface should only be used qualitatively and not quantitatively, since, in practice, there is no uniform support of the tie 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. The simplest way to calculate it is to divide the Action R on the tie by the "effective" tie seating surface:

$$\bar{p}_{ballast} = \frac{R_{max}}{F_{eff-tie}} = \frac{R_{max}}{L_{eff-tie}\cdot b_{tie}} = \frac{R_{max}}{(L_{tie}-e)\cdot b_{tie}} \qquad (11)$$

Where: R_{max} the maximum reaction/action on each tie derived from each method, F_{tie}= effective tie seating surface, L_{tie} = length of the tie i.e. 8.5 feet or 2590 mm, e = gauge of the track (~1500 mm), $L_{eff-tie}$ calculated from Equation (12) with the assumption that the center of the tie is unsupported, b_{tie} = width of the tie at the seating surface.

$$L_{eff-tie} = (L_{tie} - e) \qquad (12)$$

The load on the ballast-bed should be equal to the sum of the mean (static) load + 1 to 3 standard deviations (probability of appearance/level of confidence P = 68.3% ÷ 99.7%) depending on the circulation speed and the necessary maintenance work. This implies that the action calculated from the methods cited in German and French literature as well as Giannakos (2004) method could be reduced for a probability of appearance relevant to 68.3 % for the subgrade or 99.5 % for the ballast.

French method: Equation (2) covers a 95.5% probability of appearance, so it is used for the calculation of the stress on ballast in Equation (11) since for the calculation of the stress on the formation a level of confidence on the order of 68.3 % to 95.5 %should be considered..

German method: Equation (4) is used for the calculation of R_{max} in Equation (11) using t=2 (probability 95.5 %) for the stress on the ballast, and t=1 or 2 (probability 68.3 % to 95.5 %) for the stress on the subgrade.

AREMA method: This method does not use the probabilistic approach. Equation (5) is used in any case for the calculation of R_{max} in Equation (11).

Giannakos (2004) method: Equation (6) is used for the calculation of R_{max} in Equation (11), using μ=2 (probability 95.5 %) for the stress on the ballast, and μ=1 or 2 (probability 68.3 % to 95.5 %) for the stress on the subgrade and the average stress under the tie seating surface should be calculated by the following equation (Giannakos, 2004, 2010 b):

$$\bar{p} = \bar{A}_{subsidence}\cdot(Q_{wheel}+Q_{\alpha}) + \frac{2\cdot\sqrt{[\sigma(\Delta Q_{NSM})]^2+[\sigma(\Delta Q_{SM})]^2}}{h_{TR}}\cdot C \qquad (13)$$

where: $$\bar{A}_{subsidence} = \frac{1}{2\sqrt{2}}\cdot\sqrt[4]{\frac{\ell^3}{E\cdot J\cdot h_{TR}^3}} \qquad (14),$$

$$C = \frac{\rho_{total}}{\left(\frac{F_{eff-tie}}{2}\right)} \qquad (15)$$

$F_{eff-tie}$ = the effective tie seating surface (for monoblock ties the central non-loaded area should be subtracted) as in Equation (12).

For conventional superstructure, which comprises of rail, fastenings, sleepers and ballast, there is an optimum life-cycle from an economic point of view. The mean stress on the subgrade (magnitude of the pressure on the contact surface under ballast-bed) plays a major role in the maintenance needs and planning and consequently on the costs. On the basis of AASHTO testing for road construction, the following formula is valid:

Decrease in track geometry quality = (increase in stress on the ballast bed)^m (16)

where m = 3 to 4.

When the stress on the ballast-bed is increased by 10%, then a more rapid decrease in the track's geometry from 1.33 to 1.46 times occurs with a corresponding increase of the maintenance cost.

The key parameters for the definition of the track's vertical stiffness and deformation are the quality of subgrade and elastic pad, both of which characterize the subsidence (or the stiffness) of a track, that depends on the distribution of loads between the sleeper that carries the axle and the adjacent sleepers (Eisenmann, 1988, 1981, 1980). Among them it is the formation of the track that presents residual deformations: subsidences and lateral displacements, directly connected to the deterioration of the so-called geometry of the track, which can be nevertheless described much more specifically as quality of the track.

Minimizing or diminishing the subsidence practically minimizes the permanent deformation of the track. In order to achieve that, the mean pressure should be kept below a certain value.

It is imperative to reduce as much as possible the development of vertical, primarily, as well as lateral displacements on the subgrade layer. On the contrary, the total subsidence of the track structure should acquire a high value, in order to distribute the load Q_{total} at a longer distance from its acting point and consequently to a greater number of adjacent sleepers. This will minimize the action/reaction on each sleeper. The above two requirements are contradictory.

The average stress on ballast-bed was calculated applying Equation (11) for the methods cited in French, German and American literature and Equation (11) for the Giannakos (2004) method and for a 95.5% level of confidence. The results are depicted in Figure 5 for track equipped with W14 fastenings (European) and Figure 6 for for track equipped with Japanese fastenings with pads of 60 MN/m as in Figure 2. The solution is the adoption of very "soft" fastening pads resulting in a high value of subsidence due to their resilient behaviour that secures non-permanent deformation and, consequently, excellent preservation of the geometry/quality of the track.

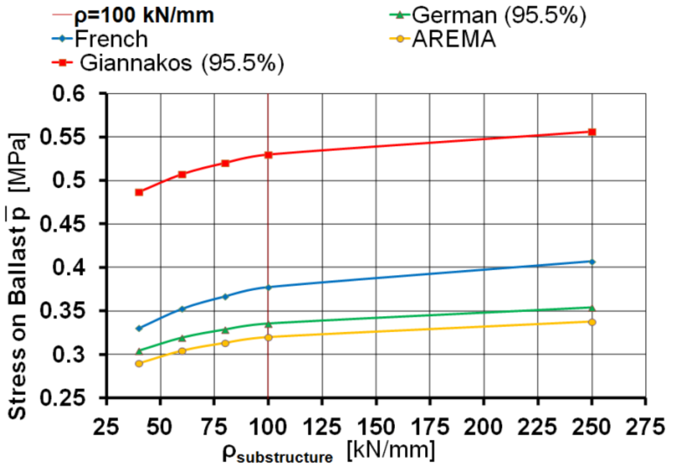

Figure 5: Stresses on the ballast-bed calculated by the four methods for fastening W14 and pad Zw700 Saargummi.

For a given quality of ballast material, as far as the ballast deformations are concerned, this is accomplished by the correct combination and usage of heavy track machinery (ballast regulator, tamping machine, dynamic stabilizer). In Giannakos (2010a) a relationship between ballast quality and life-cycle is cited. For the layers underneath the ballast a very well-executed construction is required: crushed stone material in the upper layer, with a compaction of 100% Proctor Modified or 105% Proctor Normal (Giannakos, 1999). According to the demands of the design requirement for the modulus of elasticity E_{v2} (taken from the second load step in a plate loading test) is: $E_{v2} \geq 80$ N/mm^2 (MPa) for the subgrade in the case of ballasted track. The permissible compressive stress on the formation can be established using the following equation (Esveld, 2001):

$$\bar{\sigma}_z = \frac{0.006 \cdot E_{v2}}{1 + 0.7 \cdot \log n} \qquad (17)$$

where: E_{v2} modulus of elasticity taken from the second load step in a plate loading test, n number of load cycles (usually 2 million cycles).

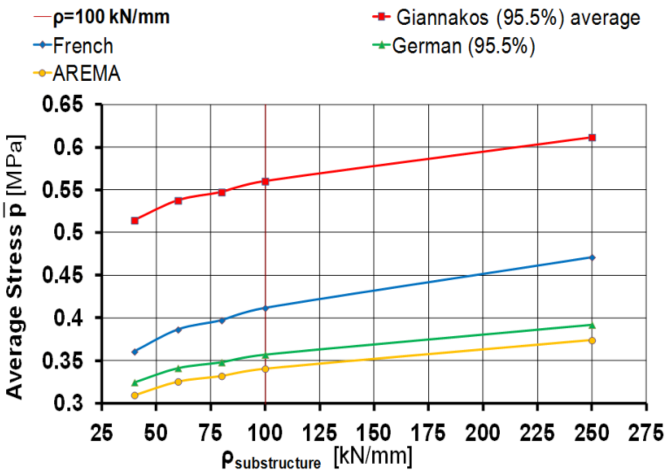

Figure 6: Stresses on the ballast-bed calculated by the four methods for Japanese fastening with pad of second kind (60 MN/m).

For 2 million cycles and E_{v2}=80 N/mm^2, then the permissible compressive stress for the formation should be (see also Esveld, 2001, p. 95, 258) :

$\bar{\sigma}_z$=0.089 MPa (18)

The stress on the subgrade, assuming a distribution cone of 45 degrees and a ballast-bed thickness of 30 cm underneath the lower contact surface of the sleeper to the upper surface of the subgrade, can be estimated as follows:

Sleeper seating surface S (length 2.60):

S_{Tie}*=1100 mm x 260 mm ≈ 285.000 mm*^{2}

Surface on the top of Subgrade:

$S_{Subgrade}$≈(2600/2+300)x600=1600 x 600⇨

$S_{Subgrade}$ = 960.000 mm^2

Relation $S_{Subgrade}$ / $S_{Sleeper}$ =3,368

Consequently $p_{Subgrade}$=$p_{Sleeper}$/3,368 (19)

And its maximum possible value is (for $\rho_{subgrade}$ = 100 kN/mm and probability of occurrence 68.3%, Giannakos method μ=1) :

max $\bar{p}_{Subgrade}$≈0.376 / 3,368 = 0.112 > 0.089 MPa

in the case of the quality of formation described above with E_{v2}=80 N/mm^2. It is worth noticing that according to the most adverse results for the case of ballasted track a formation quality of E_{v2}=80 N/mm^2 -accepted in some railway networks- could not be accepted and a subgrade's quality of at least . E_{v2}=105 MPa with a $\bar{\sigma}_z$=0.116 MPa is required during design phase. This complies with the experience obtained from the High Speed lines.

5 CONCLUSIONS

The parametric investigation performed in this paper showed that the role of the formation of the railway track (blanket layer and beneath in the case of Ballasted Track and the Frost Protection Layer in the case of Ballastless Track) is key to its overall performance therefore special attention should be paid during design and a very strict supervision during construction. For the case of the Ballastless Track an excellent quality of the top of the formation/substructure with E_{v2}=120 MPa is expected to guarantee a satisfactory performance of the track. Under the most adverse conditions in the case of Ballasted Track a quality of E_{v2}=80 N/mm^2 or 11.60 kips/in^2 is not expected to behave satisfactorily and a quality of E_{v2}=105 N/mm^2 or 15.23 kips/ in^2 or better should be required.

ACKNOWLEDGEMENTS

I would like to thank Mr Kenji Takayanagi of Mitsui and Mr. O. Haga of the JRTT for sending me the load-deflection curves of the Japanese fastenings on 2005, after a visit at Japan.

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