Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-1769

Method for estimating railroad track settlements due to dynamic traffic loads

Méthode pour l'estimation des tassements de la voie ferrée dues à l'action des charges dynamiques

Rudney C. Queiroz
Department of Civil Engineering,
Sao Paulo State University at Bauru, Unesp, Bauru (SP) Brazil

Emir José Macari
College of Science, Mathematics and Engineering,
The University of Texas at Brownsville, UTB, Brownsville (TX) USA

ABSTRACT

This paper is the result of real-scale physical modeling study designed to simulate the load-deformation characteristics of railroad foundation systems that include the railroad ties, the ballast, and the sub-base layers of a railroad embankment. The study presents comparisons of the application of dynamic loads of 100kN on the rails, and the resulting deformations during a 500,000 cycle testing period for three rail support systems; wood, concrete and steel. The results show that the deformation curve has an exponential shape, with the larger portion of the deformation occurring during the first 50,000 load cycles followed by a tendency to stabilize between 100,000 to 500,000 cycles. These results indicate that the critical phase of deformations of a new railroad is within the first 50,000 cycles of loading, and after that, it slowly attenuates as it approaches a stable value. The paper also presents empirically derived formulations for the estimation of the deformations of the rail supports as a result of rail traffic.

RÉSUMÉ

L' article présente les resultats et conclusions d'un étude realizé sur un modéle en échelle naturelle, des déformations subies par les traverses, base et couches finalles de remblai de la voie ferrée en fonction du nombre de solicitations de charge. L'étude a comparé les résultats de l'application de charge dynamique de 100 kN, sur les rails, et mésure des déformations pendant un total de 500.000 cycles de charge. Ils ont été étudiés les traverses en bois, en betón précontraint et en acier, avec un éncartement de voie de 1.600mm. Les résultats obtenus ont montré que les courbes de déformation ont un comportement exponentiel, avec une deformation plus grande dans les premiers 50.000 cycles de charge. Les déformations se sont estabilizées entre 100.000 et 500.000 cycles. Ces résultats ont permit conclure que l'étape crytique des déformations va jusqu'à environ 50.000 cycles, a partir de cette valeur, et une estabilization des déformations. L'article présente encore des formulations inédites pour l'estimation dês déformations de la voie ferrée pendant lês solicitations due au trafic ferroviaire.

1 INTRODUCTION

Each element of a railroad's permanent way experiences deformations that result from the continuous loading-unloading-reloading cycles from rail traffic. If excessive, these deformations can detrimentally affect the performance and operation of the rail system.

The rails are in essence structural elements and can be considered as beams on elasto-plastic foundations with varying mechanical properties. On the other hand, the study of the ballast, composed of course granular material (generally crushed stone), does not suffer equitable deformations as the other components of a railroad system.

Very few studies of load-deformation behavior of railroad systems are part of the technical literature. Those studies are not widely published and often present over-simplified models of the "real-life" in-situ conditions; Hantzschel (1878), Schubert (1898), Bauchal (1900), Byers (1909), Talbot et al. (1919), Rives et al. (1977), as well as the most recent, of Clarke (1957), Resende & Davidovitsch (1975), Schramm (1976), Sauvage & Richez (1978), Raymond (1978), Leshchinsky et al. (1983), Queiroz (1990) and Queiroz (2002).

Therefore, the study presented in this paper is an attempt to cover some of the gaps in the state-of-knowledge of railroad infrastructure load-deformation behavior.

The study included the building of a full-scale physical model. Loads were applied to the model's rails to approximate real loading conditions under rail traffic. The model consisted of a traverse section of real railroad infrastructure. Three different railroad ties were used in the study; one set made of steel, a second made of wood, and the third one made of prestressed-concrete monoblock. The ties were installed on a ballast group and soil compacted inside a box of reinforced concrete, wrapped up for a reaction system and application of loads.

The experimental model setup was prepared by compacting several soil layers within the reinforced concrete box. Load cells were placed between some of the soils layers. After the compacted embankment reached the desired height, a layer of crushed-stone ballast was placed over it and the railroad ties were then mounted on top.

Static and dynamic vertical loading was applied directly over the railroad ties. Stress and strain response of the ballast and compacted soil layers was measured during and after the application of the loads. The following section presents details of the stress distribution induced by the external loads.

2 STRESS DISTRIBUTION IN THE BALLAST LAYERS

The study of stress distribution within the ballast and compacted clay layers has been a topic of great interest since the development of the early railroad lines. As a result, numerous empirical and, recently, more sophisticated numerical formulations have lead to the development of standards that are widely used by the railroad industry today in, both, developed and developing countries alike. One of the main factors that is affected by the distribution of stresses within the railroad's buried infrastructure is the height of the embankment. The determination of the design height of the ballast has been directly related to the shear stress capacity of the soils that form the platform.

An empirical approach, used in the late 1800's in Germany, specified that the height of the ballast should be the same to the medium value of the separation of the outer edges of the railroad ties', plus additional 20cm. Another approach, made popular in the United States, considered that the height of the ballast should be the medium spacing among the railroad ties' axes, increased of a variable value from 7,5 to 10,0cm (Fig.1).

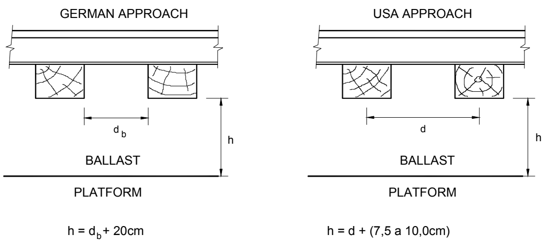

Figure 1. Simplified German and USA approaches.

These last two approaches did not take into consideration the contact stresses between the ballast and the platform. Shubert, in the late 1800's, recommended a height of 50cm for the ballast layer for loamy clay platforms, Rives et al. (1977). At the same time, Deharme, proposed a trapezoidal stress distribution below the railroad ties, Rives et al. (1977). One of the first theoretical models for the design of railroad platforms was introduced by Byers (1909). Byer's model considered the rail ballast as an overlap of cubes in order to easily obtain the stress at any given depth within the ballast.

In general, however, Europeans assumed the triangular stress distribution at 45º from the vertical axis while in the United States the stresses were assumed to be distributed at 30º from the vertical axis. Hence, the stresses at any point within the ballast and platform could be evaluated by calculating the new effective area over which the loads were distributed.

Milosevic, was the first one to consider stress superposition and used his methodology to properly assess the distance that should be used between individual railroad ties, Rives et al. (1977). As noted above, the original contact stress under the railroad ties get redistributed within the ballast. Most railroad designers assumed the simplified linearly increasing triangular stress distribution. The angle of distribution from the vertical is designated α. The upper limit of α, for the case of angular dry crushed stone ballast, is approximately 41º. The lower limit for the case of fine moist sandy ballast is approximately 30º. Thus, as medium value many have considered α to be 36º (Schramm, 1977).

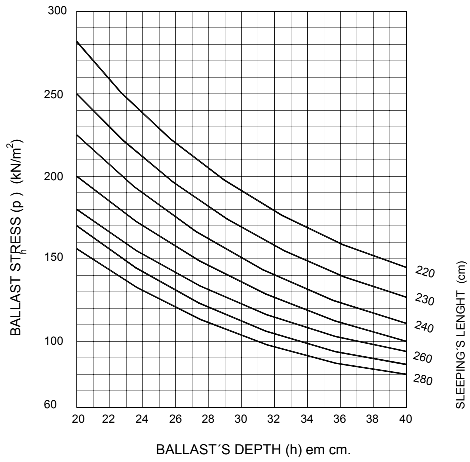

Figure 2. Pressure of ballast as a function of the height of the ballast and of the distance between railroad ties (Source: Schramm, 1977).

Therefore, Schramm (1977) stated that the stresses within the ballast depend mainly on the length the railroad ties (t) and of the height (h) of the ballast. Variations in the railroad ties width appear to have very little effect (Fig.2).

Figure 3 was developed using the finite element method for a ballast height of 60cm, for a spacing between railroad ties of 50cm and for three values of the angle of internal friction ϕ = 10º, 20º and 30º, as a function of depth (Robnett et al. (1975). The values used for the angle of internal friction for the ballast are relatively low. However, Fig. 3 demonstrates the influence of the frictional characteristics of the soil in the stress distribution response. Triaxial tests have shown that the average values of the internal friction angle for crushed stone (commonly used as ballast) vary between 35 and 40º (Leps, 1970).

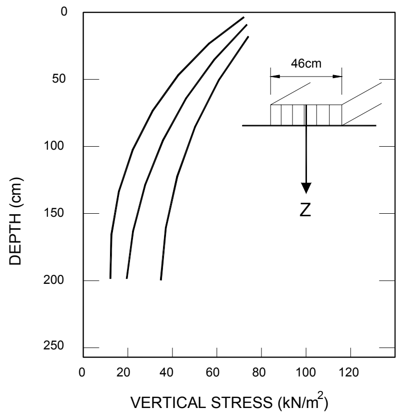

Figure 3. Vertical Stress Distribution within the ballast layer from finite element analyses (Source: Robnett, 1975).

Eisenmann and Kaess, *apud* Macedo (1982), using Boussinesq's theory of a semi-infinite space obtained a graph where σ_z represents the vertical principal stress, σ_x represents the horizontal principal stress, ν_o stress under tie, and τ_{xz} is the shear stress acting on "x" in the "z" direction (Fig.4).

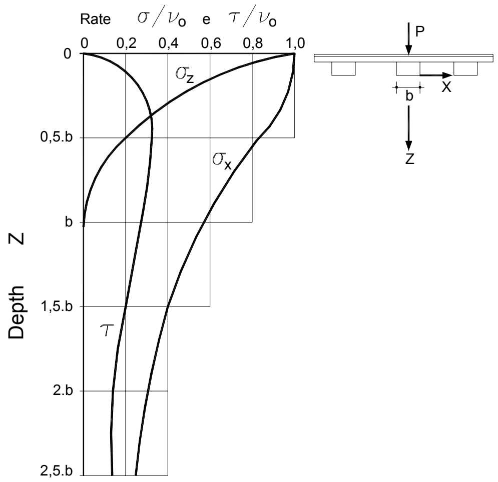

Figure 4. Eisenmann and Kaess hypothesis.

3 MATERIALS AND METHODS

The tests were performed on a full-scale physical model, inside a steel-reinforced concrete box that was used as a reaction frame to the soil pressures. That model attempted to represent realistic in-situ conditions and loads of a railroad. The soils used where those commonly used in Brazil as ballast and platform materials. The reinforced concrete test box was 3,7 x 1,3 x 1,0m, in length, width and height, respectively. Preliminary finite element analyses showed that the boundary effects would be minimal with these dimensions. The test soil, characterized as a medium to fine sandy clay, was placed and compacted in two 25cm layers, following rigorous compaction control methods, to a final height of 50cm.

As the soil was being compacted, deformation and load monitoring instruments were position within the platform to a depth of 30cm. After compaction of the clayey soil layers, a transition layer of gravel of 1.5cm was placed, followed by a 2.5cm layer of #1 stone. Finally, a 30cm layer of ballast material was rained on top of the transition layers. After each test series, the ballast material was completely removed and a fresh layer was placed prior to the next test series. The ballast material was composed of crushed granitic rock, commonly used for railroad ballast. The ballast was substituted with fresh material at the end of each test series to ensure that the crushed rock had not experienced further crushing, hence changing its engineering properties. The deformation readings taken at the ballast-soil contact were compared to the surface reference elevation to assess the surface deformations of the platform soils.

The railroad ties used in this study were: (1) steel laminate type UIC 865-1, (2) wood railroad tie, (Hymenaea courbaril L.), in the standard of 2.80 x dimensions 0.24 x 0,17m, length, width and height, respectively, and (3) prestressed concrete monoblock used by railroads in the State of Sao Paulo. The railroad ties used had dimensions of 1600mm for the wide gauge. The railroad ties were installed with two segments of rail TR-68, for the application of the static and dynamic loads. For application of the loads, two 500kN capacity hydraulic jacks were used.

The jacks were attached to the lower portion of the reaction reinforced concrete, in order to evenly apply the loads to the rail segments, exactly in the vertical direction.

The two rail segments were subjected to vertical downward sinusoidal dynamic loads (down) varying between 5 and 100kN. 100kN was used as the maximum load because of the following considerations:

a) wheel load: P = 150kN (load train),
b) coefficient of load distribution for the railroad tie: $\alpha_1 = 0.5$ (medium value calculated for rail TR-68 and the conditions of the road).
c) dynamic coefficient: $\phi = 1.30$ (method of the AREA for V = 90km/h).

Two cycles per second was considered to be the loading frequency corresponding to a train travelling at 80 km/h. The loading periods were applied at intervals as shown below.

Static test Cycle number (N)
1 - - - - - - - - - - - - - - - - - 0 (before beginning the tests)
2 - - - - - - - - - - - - - - - - - 25.000
3 - - - - - - - - - - - - - - - - - 50.000
4 - - - - - - - - - - - - - - - - - 100.000
5 - - - - - - - - - - - - - - - - - 150.000
6 - - - - - - - - - - - - - - - - - 225.000
7 - - - - - - - - - - - - - - - - - 300.000
8 - - - - - - - - - - - - - - - - - 400.000
9 - - - - - - - - - - - - - - - - - 500.000

4 RESULTS OF MODELING STUDY

After the application of each interval of dynamic loading, surface deformations were measured below each railroad tie. The measurements were done during the application of a constant static load. Table 1 shows the values of the deformations as a function of the cycle number, for a maximum load of 100kN.

Table 1: Deformations vs. cycle number

DEFORMATIONS (mm)			CYCLE NUMBER (N)
CONCRETE	TIMBER	STEEL	
2,38	7,80	12,13	0
4,67	14,30	18,07	25.000
5,05	15,42	19,68	50.000
5,73	15,92	20,28	100.000
6,39	16,59	20,60	150.000
6,47	17,35	21,20	225.000
6,48	18,09	21,42	300.000
6,85	18,67	21,85	400.000
7,07	19,17	22,26	500.000

Figure 5 graphically presents the results shown in Table 1, as curves of strain as a function of cycle number for the three types of railroad ties; prestressed concrete, wood and steel.

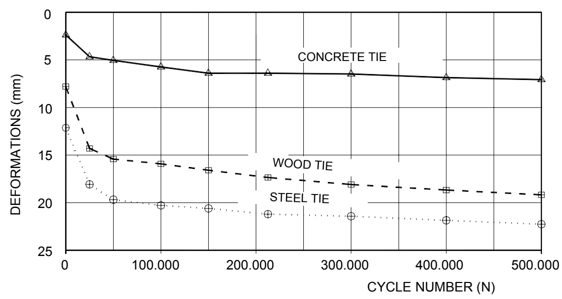

Figure 5. Deformation vs. cycle number for the three types of railroad ties prestressed concrete, wood and steel.

5. ANALYSIS OF THE RESULTS

The prestressed concrete railroad ties underwent less deformation than the wood or steel ties under the same loading conditions. This result can be attributed to the higher stiffness of the concrete ties that resulted in a more uniform stress distribution in the ballast-soil interface. The wood and steel railroad ties experienced similar deformations, (the steel ties experienced approximately 5mm deformations larger than the wood railroad ties). The deformation trends of all three types of railroad ties show similar exponential behavior. A regression analysis yielded the following empirical exponential curve:

$$y = a + bx + cx^{0,5} \qquad (1)$$

Substituting the resulting constants results in (N is number of loading cycles):

a) Prestressed Concrete Monoblock Railroad Tie:

$$D = 4,29 - \frac{N}{503414} + \frac{\sqrt{N}}{187,5} \quad \text{(mm)} \qquad (2)$$

b) Wood Railroad Tie:

$$D = 13,94 + \frac{N}{501577} + \frac{\sqrt{N}}{162,5} \quad \text{(mm)} \qquad (3)$$

c) Steel Railroad Tie:

$$D = 17,67 - \frac{N}{230021} + \frac{\sqrt{N}}{111,1} \quad \text{(mm)} \qquad (4)$$

Hence, under similar conditions, the above equations may be used for the assessment of expected deformations (D) that railroad infrastructure would suffer under typical traffic.

6 CONCLUSIONS

The deformation suffered by the wood and steel railroad ties showed a similar behavior, with ≅4mm higher values for the steel railroad tie.

The prestressed concrete railroad tie showed a better performance than the other two ties, resulting in maximum deformations of 7,07mm after 500,000 load cycles.

The analysis of the deformation vs. number of load cycle curves showed an exponential behavior that slowly reaches a stable value without further deformation. The exponential regressive analysis of the three curves resulted in representative equations of the deformations as a function of the number of load cycles. These equations are the most important product of this study.

This study represented a first in Brazil. It has been shown that full-scale, properly instrumented and monitored studies of railroad infrastructure can provide a wealth of knowledge that will not only help for the design and construction of new railroad tracks but also help in the revamping of the current railroad infrastructure of Brazil.

ACKNOWLEDGEMENTS

The authors would like to express their sincere thanks to Fapesp (*Fundação de Amparo à Pesquisa do Estado de São Paulo*) for the financial support for the post doctorate fellowship of the author that made possible the preparation of this article. In addition, the authors would also like to thank the IPT (*Instituto de Pesquisas Tecnológicas do Estado de São Paulo*) and Department of Civil Engineering of Sao Paulo State University, Unesp at Bauru (SP), Brazil, and the Department of Civil and Environmental Engineering at Louisiana State University, LSU at Baton Rouge (LA), USA.

REFERENCES

Clarke, C.W. (1957) Track loading fundamentals. The Railway Gazette, Vol. 106, Part 1, Jan. 11, p. 45-48, Part 2, Jan. 25, p. 103-107, Part 3, Feb. 8, p. 157-160 & 163. Part 4, Feb. 22, p. 220-221, Part 5, Mar. 8, p. 274-278, Parte , Mar. 22, p. 335-336, Part 7, April. 26, p. 479-481.

Leps, T.M. (1970) Review of shearing strength of rockfill. Journal of the Soil Mech. and Found. Div., Proc. of the ASCE, July 1970, p. 1159-1170.

Leshchinsky, D. et al. (1983) A simplified methodology to evaluate the effect of heavier axle loads on track substructure performance. Research & Test Department Assoc. of American Railroad, USA, p. 471-496.

Macedo, C. (1982) A influência do material de lastro na conservação da via. Subcomitê Brasileiro de Estudos Gerais-ABNT (Associação Brasileira de Normas Técnicas), São Paulo, SP, 32p.

Queiroz. R.C. (1990) Estudo experimental de tensões e deformações em camadas da infra-estrutura e superestrutura ferroviária. D.Sc. thesis, USP, Universidade de São Paulo, São Carlos, Brazil, 223p.

Queiroz, R.C. and Gaioto, N. (1994) Stresses and strains in ballast: an experimental approach. XIX Pan-American Railroad Congress, Isla de Margarita, Venezuela, nº 35, Comision I, Jan/1994, 12 p.

Queiroz, R.C. (2002) Contribuição ao estudo experimental das resistências unitárias horizontais e módulo de deformação, em modelo de via permanente ferroviária. Thesis, Civil Eng. Department, Unesp, Universidade Estadual Paulista, Bauru, Brazil, 134 p.

Raymond, G.P. (1978) Soil-structural interaction and concrete tie design. Journal of the Geotechnical Engineering Division. American Society of Civil Engineers, Vol. 104, nº GT 2, Feb. 1978, p. 676-681.

Resende Filho, S.P. and Davidovitsch, A.D. (1975) Tentativa de dimensionamento dos terraplenos ferroviários considerando os efeitos repetitivos das cargas móveis. ENGEFER–RFFSA. Rio de Janeiro, Brazil, 20p.

Rives, F.O. et al. (1977) Tratado de ferrocarriles. Editora Rueda, Vol. I, Madrid, Spain, 692p.

Robnett, Q.L. et al. (1976) Development of a structural model and materials evaluation procedures. Ballast and foundation research. Transportation Research Laboratory, Department of Civil Engineering, University of Illinois at Urbana Champaign, Urbana, Il, USA, 76p.

Sauvage, R. and Richez, G. (1978) Les couches d´assise de la voie ferré. Revue Générale des Chemins de Fer, Dez. 1978, Paris, p. 773-795.

Schramm, G. (1976) Técnica e economia na via permanente ferroviária. Edited by RFFSA, Rio de Janeiro, Brazil, 297p.