Features of Rail Strength Analysis in Conditions of Increased Force Loading

G. Guramishvili, M. Moistsrapishvili, L. Andghuladze

Abstract—In the article are considered the problems arising at increasing of transferring from rolling stock axles on rail loading from 210 KN up to 270 KN and is offered for rail strength analysis definition of rail force loading complex integral characteristic with taking into account all affecting force factors that is characterizing specific operation condition of rail structure and defines the working capability of structure.

As result of analysis due mentioned method is obtained that in the conditions of 270 KN loading the rail meets the working assessment criteria of rail and rail structures: Strength, rail track stability, rail links stability and its transverse stability, traffic safety condition that is rather important for post-Soviet countries railways.

Keywords—Axial loading, rail force loading, rail structure, rail strength analysis, rail track stability.

I. INTRODUCTION

RAILWAY transport includes many closely interconnected parts that create a unified economic system. As this system responsible and main point is presented the railway rail.

The structure of railway rail would provide the safe and uninterrupted trains traffic on the railway track determined speeds, the whole year round and round-the-clock, in arbitrary weather conditions.

The railway rail represents a unified structure, all elements of that are behavior in harmony. Due to the extremely large value of transferred static and dynamic loads from rolling stock on rail's superstructure elements, the material selection for such elements has great importance. It should be noted that the non-matching work track of rail's superstructure arbitrary elements completely violates the operating conditions of railway track as a unified structure. The matching work of elements lies in the fact that the previous element transfer to the next one such values of loads that this element's material would undergo and does not exceed the limit of his strength [1].

To illustrate the rail's superstructure behavior as a unified structure is sufficient to consider vertical load transfer scheme from rolling stock to rail's superstructure and from rail's superstructure to subgrade (Fig. 1).

Transferred from rail rolling stock wheel dynamic load in range from 100 KN up to 260 KN in wheel-rail contact point

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makes the contact stress magnitude up to 900MPa. While the bending stresses magnitude in the rail base rib varies in the range 100-200 MPa [2].
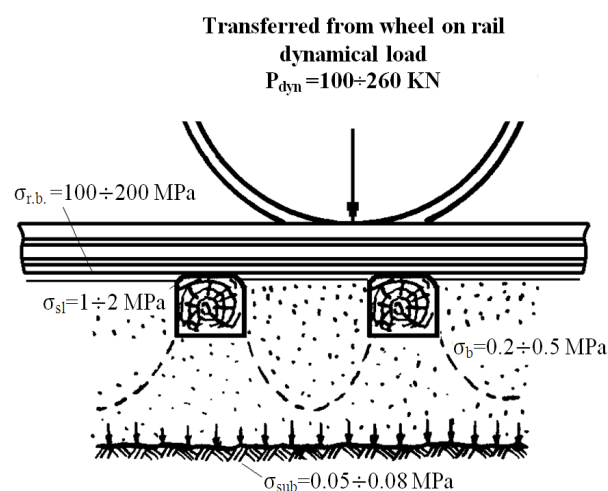

Fig. 1 The vertical load transfer scheme from the rolling stock on rails

Due the multiple impacts of transferred from train dynamic loads and natural factors forces the rail constantly is being in mode of deformation and in it accumulates the residual deformations. Even the elementary violation in rail structure operation conditions would cause the violation of train traffic safety condition that is unacceptable, because the providing of traffic safety represents main requirements of railway transport operation [3].

As it was mentioned the railway track was transferred large value of dynamic loads that stipulates the stressed work of track. The track stressed work is accompanied by systematic origination of elastic and residual deformations. The assuming of residual deformation in the structure's working conditions, except to railway track other engineering facilities are very rare.

The origination of elastic and residual deformations requires the systematically execution of track maintenance and repair works.

The accumulation residual deformations in train movement conditions (rails deterioration, sleepers rotting and mechanical damage, ballast contamination, roadbed disease, etc.) becomes more and more intensely in direct proportion to time factor.

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II. FEATURES OF RAIL STRENGTH ANALYSIS IN CONDITIONS OF INCREASED FORCE LOADING

During long time in the Georgia railway network the transferred from rolling stock axle on rail load makes up to 210 KN. in recent decades as a result of increasing rail-cars load carrying capacity the mentioned load has reached up to 250 KN. Due the increasing in axle loads is increased crude rail failures, especially with contact - fatigue defects, is decreased service life of track all elements, the track correction works is increased near about in 1.4 times. The integration of Georgia transport system in the world the transport system as the shortest and most convenient TRACECA transport mail line between West and East makes on the agenda the traffic of such rolling stock on that applied on axle value of loading make up to 270 KN [4]-[5].

The rail superstructure strength and stability analysis is based on such theory of bending, according to which the rail is considered as a continuous infinite length beam, located on total elastic basis. The calculations are based on probability theory and mathematical statistics regularities, because the acting on the rails forces are versatile and variable.

According to this theory the acting on the rail dynamic moment, the transferred on sleepers load and rail elastic deflection value will be calculated dye the following formulae

$$M_{din} = \frac{f}{4k}\left(P_{av} + 2.5S + \sum P_{av}\mu\right) \qquad (1)$$

$$Q_{din} = \frac{k\ell}{2}\left(P_{av} + 2.5S + \sum P_{av}\eta\right) \qquad (2)$$

$$y_{din} = \frac{k}{2U}\left(P_{av} + 2.5S + \sum P_{av}\eta\right) \qquad (3)$$

Accordingly the stress in rail base rib due the dynamic moment is equal to

$$\sigma_{r.b.} = \frac{M_{din}}{W} = \frac{f}{4kW}(P_{av} + 2,5S + \sum P_{av}\mu) \qquad (4)$$

and value of rail head rib stress is calculated by the formula

$$\sigma_{r.h.} = \frac{\sigma_{r.b}}{f}\left\{\frac{z_h}{z_b} + (f-1)\times\frac{b_h}{b_b}\right\} \qquad (5)$$

The stress value under sleepers' bottom is equal to

$$\sigma_{sl} = \frac{Q_{din}}{w} = \frac{kl}{2w}(P_{av} + 2,5S + \sum P_{av}\eta) \qquad (6)$$

Under the sleeper in ballast layer the stresses maximum values are calculated by the formula

$$\sigma_{ball} = \frac{Q_{din}}{\Omega} = \frac{kl}{2\Omega}(P_{av} + 2,5S + \sum P_{av}\eta) \qquad (7)$$

Due the influence of rolling stock, for more accurately assessment of rails failures generation intensity, for optimization of track maintenance works is necessary to define a value of rail real force loading characteristics. One of such most important characteristics represents "reduced freight traffic density" that provides rail operation (train traffic real speed, applied on rolling stock axles load, train weight, rolling stock type, ratio of freight and passenger traffic at station, carrying tonnage) and structural (rail track plan and profile, type of rail superstructure, etc.) features on district.

At the present stage of science and technology development it is necessary to have a single generalized method that gives the possibility to adequately assess rail structure's real force loads level in conditions of its strength analysis and durability calculation. Also is necessary to have a unified methodology for determining these criteria.

Such common criteria and unified methodology for their determination, moreover, is necessary in condition of inter-state cooperation, for development of common technical requirements for rail track structure and for such mutually agreed projects, for example TRACECA Europe - Asia international transport corridor [6].

For determination of rail real force load characteristics level, the transfer coefficients were determined on the impact of freight rail-cars at 100 km/h case.

The main difference of this method in comparison of existing method represents due rail axial load and force factors impact the determining of rail track qualitative dependency level.

The district reduced freight traffic density T_{red} - mln.br.t.km/km year, will be determined by the formula

$$T_{red} = T_{(Q,q,v)}Ck \qquad (8)$$

where $T_{(Q,q,v)}$ is the design freight traffic density with taking into account the train traffic density (train weight, type of rolling stock, traffic speed, axial loading) mln.br.t.km/km. year and is equal to

$$T_{(Q,q,v)} = a_{fr}T_{fr} + a_{pas}T_{pas} \qquad (9)$$

where T_{fr}, T_{pas} are the district freight traffic density depended on according freight and passenger traffic; a_{fr}, a_{pas} are the transfer coefficients for freight and passenger trains that takes into account the influence of freight, passenger rail-cars and locomotives, as well as influence of traffic speeds on level of rail failure level; C is the coefficient, considering rail track specific district local operational conditions. In the calculations will be considered type of rail superstructure, service life, plan and profile, carrying tonnage, number of switches; k is the coefficient, considered accepted rail track geometry deviations values, depending on steady speed value for train traffic on given district.

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As the requirements for rail's ongoing maintenance works are depending on the defined by order value of speed on Georgia railway, the coefficients a_{fr} and a_{pas} are determined by these speeds.

The coefficient C is defined as average value of defined for given district coefficients:

$$C = \frac{1}{n}\sum_{n=7}^{i=1} c_i \qquad (10)$$

where C_i is the characteristic of local conditions coefficient. c_1 is the coefficient considering the rails and sleepers type, kind of ballast and its contamination level; c_2 is the coefficient considering continuous welded rail behavior; c_3 is the coefficient considering longitudinal profile influence and is determined by the formula

$$c_3 = \sum_{i=1}^{n} c'_{3i} \qquad (11)$$

c'_{3i} is the corresponding to longitudinal profile i element coefficient that will be defined due the ratio of element slope length on relative element length to the total length of profile L_i / L_{dis}.

As the district length will be accepted the railway subdivision district length, or the length of that part for which is determined the expanses of employees engaged on track maintenance works, or the interval between repairs. The impact of rise and slope will be equally taken into account.

c_4 is the coefficient considering rail track plane features and will be defined by the formula

$$c_4 = \sum_{i=1}^{n} c'_{4i} \qquad (12)$$

c'_{4i} is the coefficient corresponding to the i radius of curve, will be determined depended on curve R radius and curve length district total length ratio I_i / L_{dis}.

On Georgia railway stations often occurs in composite curves whose radius varies in the wide range. In this case as design will be accepted the smallest radius.

c_5 is the coefficient considering carrying tonnage ratio and will be defined by the following formula:

$$c_5 = \sum_{i=1}^{n} c'_{5i} \qquad (13)$$

c'_{5i} is the corresponding to i district coefficient; will be determined by the carrying tonnage and i district length ratio to the total length I_i / L_{dis}.

c_6 is the coefficient considering rail superstructure construction service life and will be defined by the formula

$$c_6 = \sum_{i=1}^{n} c'_{6i} \qquad (14)$$

c'_{6i} is the corresponding to i district coefficient; will be determined by ratio of rail superstructure service life and i district length ratio on the total length I_i / L_{dis}. c_7 is the coefficient considering number on i km of track of switchers number. k is the coefficient considering track maintenance permissible deviations values from standard, depending on traffic speeds.

In accordance with the above mentioned methods, it haa been carried out the strength for P65 type rail with "Pandrol" fasteners reinforced concrete and wood sleepers with tie cotter clamps. In the case of reinforced concrete sleeper the calculation was carried out for straight district, curves with radiuses: R=1000m, R=700m, R=500m, R=350m, R=250m (in the case of rail basis modulus of elasticity U=80MPa, U=100MPa, U=120MPa, U=140MPa, U=160MPa, U=180MPa, respectively), while additional in wooden sleepers case R=175 meters (in the case of rail basis modulus of elasticity U=27÷29.5 MPa, U=35 MPa, U=40 MPa, U=50 MPa) [6].

Was defined the values of stresses from dynamical moment in rail base rib σr.b.for the cases of reinforced concrete (Fig. 2) as well as wooden sleepers (Fig. 3).
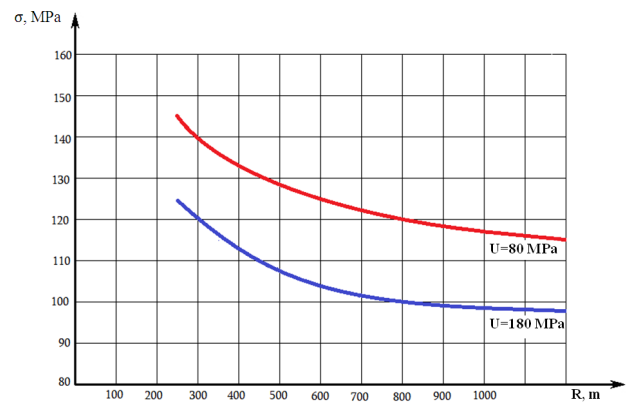

Fig. 2 Values of stresses from dynamical moment in rail base rib σr.b.in the reinforced concrete sleepers case (in summer U= 80 MPa, in winter U= 180 MPa), depending on the radius of curve
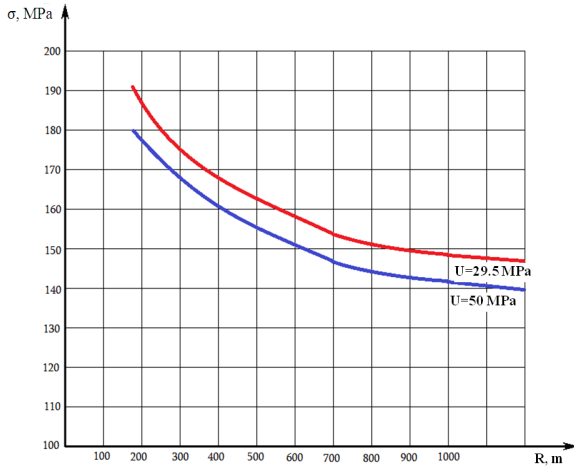
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Fig. 3 Values of stresses from dynamical moment in rail base rib σr.b.in the wood sleepers case (in summer U= 27÷29 MPa, in winter U= 50 MPa), depending on the radius of curve

The values of stresses in rail base rib σr.b. accordingly of rail base modulus of elasticity and the radius of curve in the case of reinforced concrete sleepers (varies in the range of 90,6-146,2 MPa) as well as in the case of wooden sleepers (varies in the range of 135,2-190,9 MPa) and does not exceed the permissible value [σr.b.]=200 MPa.

The values of stresses under the sleepers bottom σsl, in the wooden sleepers cases (Fig. 4), accordingly of rail base modulus of elasticity and curve radiuses (varies in the range of 0.89-1.39 MPa) and does not exceed the permissible value [σsl] =2.4 MPa.
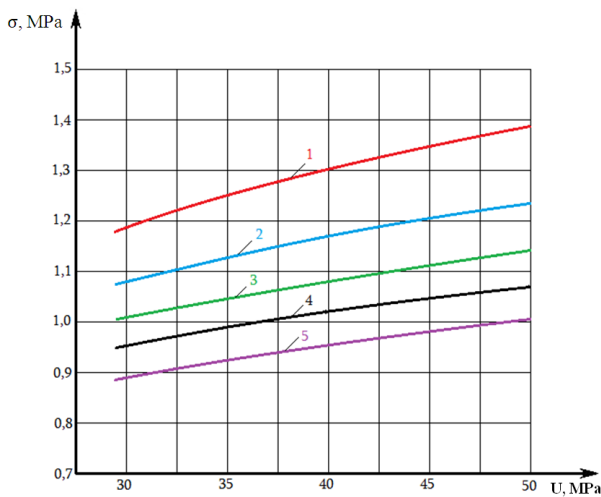

Fig. 4 Values of stresses under the sleepers bottom σsl, in the wooden sleepers cases depending on the rail base elastic modulus and the radius of curve: 1 – on straight ; 2 – R=1000m; 3 – R=350m; 4 – R=250m; 5 – R=175m
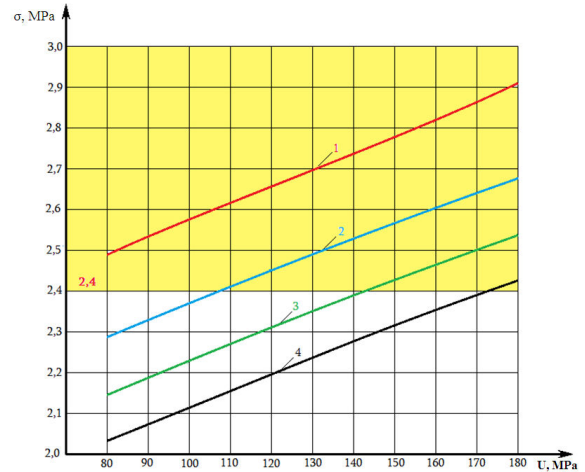

Fig. 5 Values of stresses under the sleepers bottom σsl, in the reinforced concrete sleepers cases depending on the rail base elastic modulus and the radius of curve: 1 – on straight ; 2 – R=1000m; 3 – R=350m; 4 – R=250m; 5 – R=175m

Stresses values on sleepers under the rail σsl, in the case of reinforced concrete sleepers (Fig. 5) accordingly of rail base modulus of elasticity and curve radiuses (varies in the range of 2.04-2.91 MPa) that is partly exceeds the maximum permissible value [σsl] =2.4 MPa.

Stresses values in ballast (in case of crushed stone ballast) layer σb in the case of wooden sleepers (Fig. 6), accordingly of rail base modulus of elasticity and curve radiuses (varies in the range of 0,186-0,288 MPa) does not exceed the permissible value [σb] =0.325 MPa.
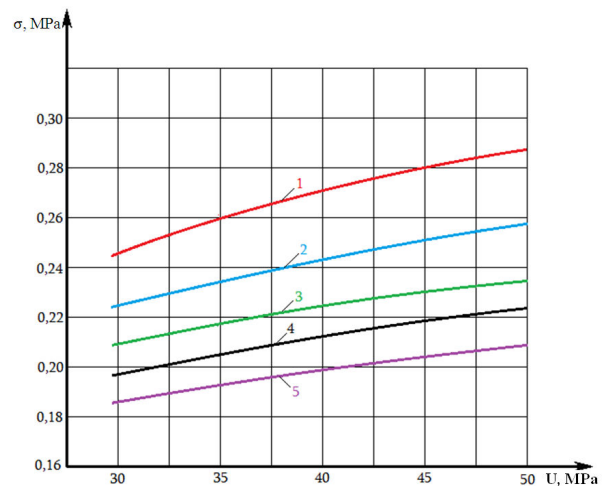

Fig. 6 Values of stresses in ballast (in case of crushed stone ballast) layer σsl, in the wooden sleepers cases depending on the rail base elastic modulus and the radius of curve 1 – on straight; 2 – R=1000m; 3 – R=350m; 4 – R=250m; 5 – R=175m

Stresses values in ballast (in case crushed stone ballast) layer in the case of reinforced concrete sleepers (Fig. 7) according to the rail base modulus of elasticity and curvature radiuses (varies in the range of 0,23-0,327 MPa) that partially exceed the permissible value [σb]=0.325 MPa.

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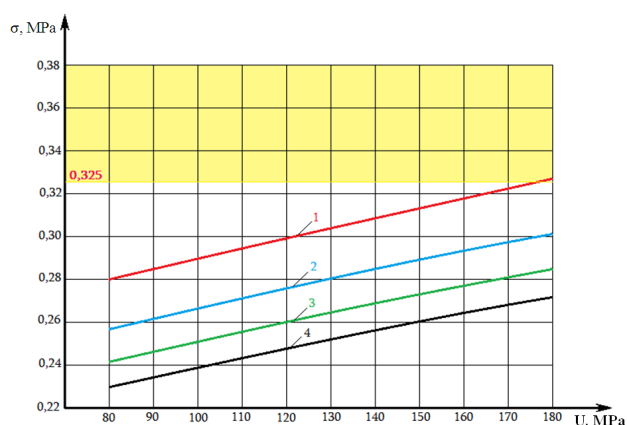International Journal of Civil and Environmental Engineering
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Fig. 7 Values of stresses in ballast (in case of crushed stone ballast) layer σsl, in the reinforced concrete sleepers cases depending on the rail base elastic modulus and the radius of curve 1 – on straight; 2 – R=1000m; 3 – R=350m; 4 – R=250m

The exceeding of design stresses on values of permissible stresses values indicates on that is necessary reinforcement of rail structures and its maintenance works. Simultaneously would be considered the exceeding of design stresses in sleepers and ballast in comparison with permissible no more than 30% (in our case in reinforced concrete sleepers makes up to 21% and at application of crushed stone ballast in reinforced concrete case approximately makes up to 1%), does not requires immediately slow up in traffic speed.

III. CONCLUSION

As result of calculation is obtained that in the condition of 270 KINEMATICS loading the rail track satisfies rails and rail structures behavior assessment criteria: strength, rail track stability, rail parts state and their transverse stability, trains safety conditions.

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