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Selection of railway track superstructure design: Modern outlooks

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ABSTRACT The paper analyses the reasons for the current opinion that a higher mass of superstructure elements is necessary for railway track sections with higher traffic density, speed and axial loads. With the help of known theories and the new one developed by the author that takes into account the time factor, it is proved that in terms of all the most significant technical indicators the best solution within the realistic limits is to use lower-mass rails and reinforced concrete sleepers. The most essential argument in favour of the smaller mass of rails per unit length is the increased track stability produced by longitudinal compressive forces in rails. A reinforced concrete sleeper of a smaller mass has been proposed and tested that increases the resistance to shear in ballast across the track axis at least two times. For stable pressing of rails to sleepers by intermediate fasteners, elastic clips should be made of plate steel instead of bar steel. It is proposed to make a plate clip shaped as a 'fish-bellied beam'. Specific examples of the proposed solutions for rails, reinforced concrete sleepers, and intermediate fastenings are given.

KEYWORDS: construction of superstructure elements; mass of superstructure elements; rails; reinforced concrete sleepers; intermediate fastenings; elastic clips; methods for the design of superstructure elements; strength; stability

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Научная статья

Выбор конструкции верхнего строения железнодорожного пути: современные представления

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АННОТАЦИЯ Анализируются причины бытующего мнения о необходимой более высокой массы элементов верхнего строения для участков железнодорожного пути с повышенными грузонапряженностью, скоростью и осевыми нагрузками. С помощью известных теорий и вновь разработанной автором, учитывающей фактор времени, доказывается, что по всем наиболее существенным техническим показателям лучшими в реальных пределах являются меньшая масса рельсов и железобетонных шпал. Значимым аргументом в пользу меньшей погонной массы рельсов служит повышение устойчивости колеи при действии в рельсах продольных сжимающих сил. Предложена, изготовлена и испытана железобетонная шпала меньшей массы, создающая при этом не менее чем в два раза повышенные сопротивления сдвигу в балласте поперек оси пути. Для стабильного прижатия рельсов к шпалам промежуточными скреплениями упругие клеммы должны быть выполнены не из прутковой, а из пластинчатой стали. Пластинчатую клемму предлагается изготовить в виде «балки равного сопротивления». Приводятся конкретные примеры предлагаемых решений по рельсам, железобетонным шпалам, промежуточным скреплениям.

КЛЮЧЕВЫЕ СЛОВА: конструкция элементов верхнего строения пути; масса элементов верхнего строения пути; рельсы; железобетонные шпалы; промежуточные скрепления; упругие клеммы; методы расчетов элементов верхнего строения; прочность; устойчивость

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INTRODUCTION

There is a notion that due to higher train velocity, traffic density and axial load levels that are typical of modern rail transport, the track superstructure should be built of heavier rails and larger reinforced concrete sleepers and, if this is not enough, also have a reinforced concrete ballastless under-rail base (a slab).

This opinion is based on works of Professor Georgy M. Shakhunyants [1–3] and the dependencies of mass of a rail per unit length on train velocity, traffic density, and axial load. These were derived fairly long ago by the approximation of trends in global engineering policies in the subject area in the mid-20th century. In his paper [1], the author presents the dependencies with a reservation that these should only be used *"as preliminary and tentative guide values"*. However, his progeny often used the formulas $Q(T, P, V)$ [3], where Q is mass per unit length; T is traffic density; P is axial load; and V is velocity, as if they were deterministic functions.

The publication [1] contained the following formulas [3]

$$Q \geq a(1 + (T_{\max}/\lambda_p)^{1/4}[(1 + 0.012V)P]^{2/3}_{\max}, \quad (1)$$

where $a \approx 1.20$ for railcars and $a = 1.13$ for locomotives; λ_p is the rail quality factor which ranges from 1.5 to 2.0 for hardened rails.

The research [4] conducted by the authors on operating railway sections has shown that within the realistic limits, the relationship in $Q(T, P, V)$ dependencies is not direct but rather almost inverse. In essence, this paradox means that Georgy M. Shakhunyants determined the approximate dependencies $Q(T, P, V)$ [1] at the time when the USSR had no continuous welded rail track and therefore he did not take into account the longitudinal thermal forces occurring in rails. Longitudinal thermal forces that occur in rails in a continuous welded rail track can cause deflections that may compromise the safety of train traffic (*Fig. 1*).

Fig. 1. Typical view of a bucked track section

Experts in the subject field are well aware that longitudinal thermal forces in rails are directly related to the cross-sectional area of rails, which is a negative factor. But the larger the cross-sectional area, the greater the bending stiffness of rail, which was seen as a positive factor. In this case, they usually referred to the fact that the dimension of cross-sectional area is m², while the second moment of area used in the formulas is in the dimension of m⁴. This was mistakenly regarded as if stiffness dominated area and hence preference should be given to rails with a higher mass per unit length.

However, if we look at particular formulas that are used for determining the conditions for stability of continuous welded rail track by known methods, then under the static conditions where no impact from trains is taken into account, these formulas will be as follows [5]

$$F = 4(EIq/f)^{1/2} \text{ with } f = ql^4/415EI, \quad (2)$$

and where the impact from trains is taken into account by applying creep theory methods and hence the time factor is incorporated as well, the formulas will look as follows [6]

$$f = f_0\exp(F^2\tau/16EI\xi) \text{ with } f = ql^4/415EI, \quad (3)$$

where f and f_0 are current and original bends of rails in the horizontal plane, respectively, with stressed irregularity along the bending length l, m; F is longitudinal compressive thermal force in rails, $F = \alpha E\omega(\Delta t)$, kN; τ is time, s; E is modulus of elasticity of rail steel, MPa; I is equivalent moment of inertia in a track panel when bent in a horizontal plane, m⁴; ξ is an empirical factor of viscous resistance of ballast per unit length for displacement of sleepers transverse to the track axis, N · s/m²; $\xi = r/\dot{\lambda}$; $\dot{\lambda} = d\lambda/d\tau$ is velocity at which sleepers move in the ballast, m/s; q is resistance of ballast per unit length to displacement of sleepers transverse to the track axis, kN/m.

For the purposes of a comparative assessment of the influence of mass of a rail per unit length on the conditions for stability of continuous welded rail track, we can use the first of the expressions (2) to determine the value of resistance per unit length q required for maintaining the stability of continuous welded rail track, depending on rail cross-sectional area ω and rail second moment of area I: $q = A(\omega^2/I)$. The same dependence is obtained from the first expression (3) if we determine the value of viscous resistance per unit length ξ necessary for maintaining the stability of continuous welded rail track depending on rail cross-sectional area ω and rail second moment of area I: $\xi = B(\omega^2/I)$.

A and *B* are the factors that incorporate all the other similar mechanical characteristics of the variants compared. The calculation shows that with a 10 kg increase in the mass of a rail per unit length, the resistance per unit length q and resistance per unit length ξ that are required to maintain the stability of continuous welded rail track should be approximately 15 % higher. The calculations assume the actual, most unfavourable states of a track with the minimal rail pressing force on the sleeper in intermediate rail fastenings and the same rail bottom width of 150 mm within the range of the rail mass per unit length of 55 to 75 kg/m. A 15 % increase in the required resistance per unit length q or ξ with an increase of the rail mass per unit length by 10 kg is a huge value that should not be neglected. Since q and ξ values are random with a large dispersion and they are difficult to control in field, values to be taken into account in calculations should be as low as possible. It should be noted that the second expressions (2) and (3) show that an in-plan stressed irregularity in rails of higher mass is less steep, which should not be regarded as their benefit.

RAILWAY TRACK SUPERSTRUCTURE

According to the existing standards, track measuring means must, first of all, detect stressed irregularities as the most dangerous ones. In rails of higher mass, an in-plan irregularity with the same slope may be stressed and hence is dangerous for traffic. For rails of lower mass, the same irregularity will be stress-free, i.e. not dangerous.

In Europe and Japan, rails with a mass per unit length of above 60 kg/m are not in use and these countries have no problems with providing stability for continuous welded rail tracks. In the United States, only rails with a higher mass per unit length are in use and they have problems with the provision of stability, but the United Sates have almost no passenger rail traffic.

At present, almost all rails no the major railway lines in Russia are R65 rails. For nearly forty years, new rails of other types have not been used in overhauls. Currently, it is proposed to produce R71 and R75 rails. The proposal is mainly based on the assumption that there is the dependence (1), or, more specifically, that in rails of greater mass, edge stresses, when calculated by the method currently in use, are reduced and vertical loads on the under-rail base from the rolling stock are distributed more evenly. At the same time, it was mistakenly assumed [7] that this would also provide higher stability of continuous welded rail track.

As for stability, the above evidence should make it clear that stability significantly decreases with an increase in mass of rails per unit length. When the actual conditions are taken into account, i.e. vertical subsidence of track, edge stresses in rails do not decrease but increase instead [8]. Although even for the most unfavourable conditions of interfacing between the track and rolling stock, the realistic values of edge stresses are far from the probable minimum values of elastic limit of rail steel (800 MPa) [9]. Edge contact stresses depend on the service life of rails. The higher the bending stiffness of rails, the greater contact stresses [10]. Based on calculations by the method currently in use, a 10 kg increase in the mass of rails per unit length produces an insignificant, about 6 %, reduction in vertical loads on the under-rail base. Vertical loads cause deflections in the under-rail base to grow, while the quality of design and the condition of the under-rail base have influence on the growth rate of subsidence at least an order of magnitude higher than mass of rails per unit length. Besides, vertical subsidence is less dangerous for movement of trains than horizontal in-plan irregularities produced by longitudinal compressive thermal forces.

Given that consequences of derailment due to impaired stability of continuous welded rail track are highly severe, especially for passenger trains, a requirement was introduced in the framework of Russia's current regulations[1] that rail strings should be fastened at a relatively high temperature. Depending on a climate profile, this temperature should be at least plus 25–30 °C. The expressions (2) and (3) show that stability conditions depend on longitudinal force F^2. In order to securely provide for the stability of continuous welded rail track, regulations should set even higher temperature levels for fastening rail strings. But this would increase tensile longitudinal thermal forces in rails that act at temperatures below the rail fastening temperatures for a longer period within a year than compressive forces. In absolute terms, the maximum possible tensile forces can be about two or three times as high as compressive forces depending on the climate zone. In cold months, tensile thermal forces make the gaps at the ends of rail strings open to the maximum, placing track bolts in shear, which should not be allowed according to traffic safety specifications.

The current standards[1] mistakenly suppose that end sections of rail strings are no more than 60 meters in length. In this case, the joint gaps at the ends of rail strings would not have opened to the extent that track bolts would be in shear. This is only possible when resistances per unit length to longitudinal displacements, as pointed out in[1], are 12 kN/m on a single rail length. In fact, the majority of researchers in the field have found by way of experiment that resistances per unit length to longitudinal displacement were much

[1] Инструкция по устройству, укладке, содержанию и ремонту бесстыкового пути (утверждена распоряжением ОАО «РЖД» 14.12.2016 № 2544р).

lower. For example, according to data of the Railway Research Institute (VNIIZhT) [11], their maximum values are 2.6 times as little as the average values given in1. But all engineering calculations should take into account the actual, most unfavourable conditions. The value of longitudinal displacement of the end of a rail string l at a single long-term decrease in the temperature of rails from the fastening temperature under the static conditions without taking into account the impact from trains is determined using the following formula [11]

$$\lambda = \alpha^2 E\omega(\Delta t)^2/2r, \quad (4)$$

and when the impact from trains is taken into account it is determined as follows [6]

$$\lambda = \alpha(\Delta t)(\pi E\omega t/K)^{1/2}, \quad (5)$$

where r is resistance per unit length of ballast displaced by sleepers along the track axis; K is an empirical factor of viscous resistance per unit length of ballast displaced by sleepers along the track axis, $N \cdot s/m^2$.

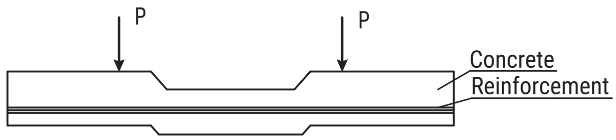
Calculations using both the formula (4) and formula (5) have shown that with actual values of r and K, displacements of rail string ends are considerably greater than the allowable gap, thus placing track bolts in shear. The larger the cross-sectional area of the rail ω, the greater the calculated values of λ (3) and (4).

In order to prevent shear failure of bolts or where shear failure has already occurred, during the cold season, maintenance personnel have to replace the rails adjacent to the ends of rail strings with expanded ones at the ends of rail strings of a continuous welded rail track. According to some unofficial data, these account for about 20 % of the total number of joints. However, this does not mean that the remaining 80 % of joint track bolts were in shear and it was just a good fortune that no shear failure occurred. Before being sheared, track bolts can accept some longitudinal tensile force; therefore shear failure of bolts is not found in every joint.

In this connection, extra-long rail strings with welded joints should be used not only within an entire railway haul, but also welded to turnouts that also have their joints welded. Any bolted joints produced by temporary repair should be removed by welding to restore the prescribed operating temperature of rail strings at such sections within a period of at least two months.

Based on the above, we propose to produce an R58 rail that will be optimal for any operating conditions in all respects, especially in terms of traffic safety.

It is believed that in order to increase the resistance per unit length to horizontal shear of reinforced concrete sleepers in crushed stone ballast, it is necessary to increase their mass. This opinion is based on the findings of experiments conducted without taking into account the impact from trains. Indeed, in accordance with Coulomb's law, under the static conditions, a greater mass of a sleeper should produce a greater resistance to its shear in ballast. However, the tests carried out on operating sections [12] have shown that horizontal longitudinal and transverse forces transmitted from the rails make sleepers with an increased mass move at least as fast as standard ones.



a

b

Fig. 2. Proposed reinforced concrete sleeper: *a* — diagram of changing the location of concrete mass and pre-stressed reinforcement; *b* — view of a sleeper laid in the track

A reinforced concrete sleeper with a mass reduced by 45 kg [13] has been proposed, fabricated and tested by a standard method both in a factory lab and on an operating track. Shear resistance in ballast transverse to the track axis it produces is at least twice as much as that of a standard one. This effect is achieved by changing the configuration of the sleeper on its supporting surface, by creating a small protrusion in the middle of it. *Fig. 2, a* shows a diagram showing the changed location of the concrete mass and pre-stressed reinforcement, and *Fig. 2, b* shows a pilot batch of the proposed sleepers manufactured based on the standard technology and laid on an operating track section.

In line with the bending moment diagram, the protrusion brings the pre-stressed reinforcement closer to the tensile region, thereby both reducing its mass, while maintaining the required strength of the sleeper, and increasing the shear resistance in ballast transverse to the track axis at least two times.

The selection of a reasonable design for intermediate fastenings is essential. All the resilient rail fastening designs currently used in Russia (ZhBR, ARS, Pandrol, Vossloh) use elastic clips made of bar steel. Bars are good for torsion loads, but they cannot be strong in bending, while it has not been possible to remove bar clips in bending in any of the known designs. As a result, when bar clips are used, the rail pressing force on the sleeper quickly goes down to zero due to plastic de-

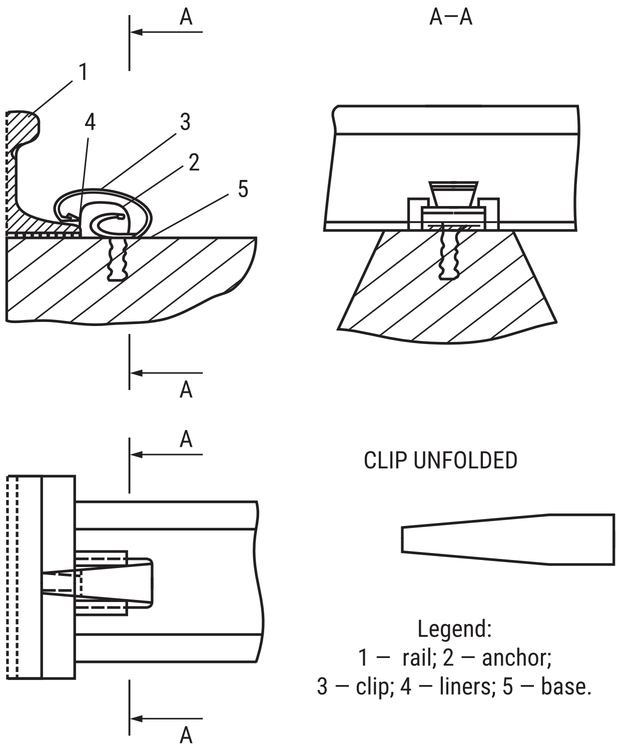


Fig. 3. Proposed boltless elastic rail fastening

formations during operation subject to traffic intensity. Then rail creeping begins.

For a clip to be strong in bending, it should be made from plate steel.

It is proposed to use a low-part fastening that is sufficiently elastic and allows for adjusting the rail position for height (within 20 mm) without sleeper tamping. A plate clip is shaped as a "fish-bellied beam" with a configuration that allows for imbedding it into the anchor [14]. This fastening is shown on *Fig. 3*.

The fastening has only one part. It is a clip inserted into an anchor embedded in a sleeper when the latter is manufactured. Unlike, for instance, ARS fastenings, this clip can be easily mounted and removed using a fairly simple mechanism. The clip is shaped as a "fish-bellied beam"; it is vandal proof thanks to a fairly high mount compression force in the anchor and has the optimal rail pressing force on the sleeper (*Fig. 3*).

Sometimes, it is proposed to use a reinforced concrete slab as an under-rail base. This increases the cost of laying the track which may then never pay off if it turns out that expenses are not expected for the current maintenance. Approximately 50 years ago in the USSR, several designs of reinforced concrete under-rail bases were tested but had to be abandoned due to the fact that with the then current maintenance, problems caused by deflections in the base were almost insoluble.

CONCLUSION

In any real-life situations, it is cheaper, easier and more reliable to use a track panel set on crushed stone ballast. At the same time, it is better to use R58 rails, the proposed lightweight reinforced concrete sleepers, and boltless elastic rail fastenings with plate clips shaped as a "fish-bellied beam".

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