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Calculation of load capacity of the main metal truss of a railway bridge

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Abstract. This paper presents calculations for mixed train schemes based on a mathematical model of moving units. Calculations of operating time were carried out considering the number of passing freight and passenger trains, the structure of freight traffic, information about the types of locomotives and loading of cars over the entire history of the bridge's operation. The methodology is based on a comparison of the durability distribution functions of elements of superstructures of real bridges and laboratory samples, tests of which were carried out at NIIZhT. Based on the test data, equations of fatigue curves and characteristics of the scatter of durability of samples were obtained, which made it possible to calculate accumulated fatigue damage and compare their results with failure cases of real elements of span structures. The main purpose of calculating the load-carrying capacity of the main metal truss of a railway bridge is the results of calculations of the classes of truss elements determined with the main and additional combination of loads.

Keywords: *mathematical models, reliability, load, bridge deck, superstructure, load-carrying capacity, endurance, stability, element class, railway bridge.*

1. Introduction

The modernization of the railway bridge is aimed at increasing the strength, bearing capacity, stability, durability and other reliability indicators of both the railway track as a whole and its components and elements that increase the life cycle, reduce the complexity and cost of maintenance of the track and obtain an economic effect during its operation.

The modernization of the railway bridge includes works that lead to a change in the category of the track, as well as to an increase in the carrying capacity of artificial structures, the ability of the track and artificial structures to carry increased axial and linear loads, a change in spatial characteristics (the plan and profile of the track, the geometry of the ballast prism, the roadbed, oversized seats), a change in the design of the track with the installation of new drainage, protective and fortifications [1].

2. Materials and methods

The calculation of the carrying capacity of the channel superstructure and the determination of the conditions for passing through it of the circulating and prospective rolling stock was carried out in accordance with the requirements of the «Guidelines for determining the carrying capacity of metallic superstructures of railway bridges» [2-3] and «Instructions for determining the conditions for passing trains over railway bridges» [3-4].

The permissible temporary load kN/m for the elements of the main truss when calculating the impact of permanent and temporary vertical loads from rolling stock with the main combination was determined by the formulas when calculating for:

endurance

$$k_n = \frac{1}{e_k \cdot n_k \cdot \Omega_k}(\chi_1 \cdot m \cdot R \cdot G - e_p \cdot p \cdot \Omega_p) \quad (1)$$

stability

$$k_y = \frac{1}{e_k \cdot n_k \cdot \Omega_k}(\chi_1 \cdot m \cdot \phi \cdot R \cdot G \mp e_p \cdot p \cdot \Omega_p) \quad (2)$$

endurance

$$k_в = \frac{1}{e_k \cdot \theta \cdot \Omega_k}(\chi_1 \cdot m \cdot \gamma_в \cdot R \cdot G - e_p \cdot p^I \cdot \Omega_p) \quad (3)$$

The permissible time load kN/m for the elements of the belts of the main trusses when calculated for a combination of vertical (permanent and temporary) and horizontal (wind and brake) loads with an additional combination was determined by the formulas when calculating for:

endurance

$$k_n = \frac{1}{e_k \cdot n_k \cdot \eta_k \cdot \Omega_k (1+\xi_T)}(\chi_1 \cdot m \cdot R \cdot G - e_p \cdot p \cdot \Omega_p - n_v \cdot \eta_v \cdot S_v) \quad (4)$$

stability

$$k_y = \frac{1}{e_k \cdot n_k \cdot \eta_k \cdot \Omega_k (1+\xi_T)}(\chi_1 \cdot m \cdot \phi \cdot R \cdot G \mp e_p \cdot p \cdot \Omega_p - n_v \cdot \eta_v \cdot S_v) \quad (5)$$

Where e_k, e_p - the proportion of vertical load from rolling stock or constant load per farm e_k=0.5; e_p=0.5,

n_k, n_v - reliability coefficients to vertical load from rolling stock and wind load, n_v=1.5 to wind load. When determining, it was considered that at λ=50m n_k = 1.10, and at $\lambda \geq$150m n_k = 1,05.

Ω_k, Ω_p - the areas of the lines of influence of axial forces in the elements of the trusses, loaded respectively by a vertical load from the rolling stock and a constant load.

χ_1- a dimension coefficient equal to 0.10 in the calculations in the SI system or 1.0 in the GHS.

m - coefficient of working conditions, m = 1.0.

R- the main design resistance of the metal, R =190 MPa.

G - estimated area of the element, cm^2. When calculating the element for axial force

G=F_0 - cross-sectional area.

$p = \sum n_{pi} \cdot p_i$ - constant load when calculating strength and stability, *kN/m.*

n_{pi} =1.1 –for the metal of the superstructure; p_i = 60 kN /m – the linear load from the superstructure [2].

n_{pi} =1.2 – for the bridge bed; p_i = 9.0 kN/m – for the bridge bed [2].

ϕ - the coefficient of longitudinal bending, (p.2.13 and Appendix 8 of the Manual [2]).

θ - a coefficient that considers the reduction of the dynamic impact of the mobile load when calculating endurance, [2].

$\gamma_в$- the coefficient of reduction of the main design resistance in calculations for wearability [2].

$p^I = \sum p_i$ - the total intensity of the normative constant load in the calculations for endurance, kN/m, [2].

n_k, n_v -coefficients of combination to temporary vertical and wind loads [2].

ξ_T – a coefficient that considers the effect of the braking load in the calculated element of the cargo belt.

S_v - axial force in the calculated element of the truss belt from the standard wind load, kN.

We will determine the impact of constant loads on the superstructure from the weight of the metal of the superstructure, the bridge bed, boxes for communications, railings, riding paths and an observation cart:

- of the weight of the metal of the superstructure, $P_м$ = 60.0 kN/m [2];
- from the weight of the bridge bed, $P_{мп}$ = 9 kN/m;
- from the weight of boxes for communications, railings, P = 20153.77 kg;
- from the weight of the consoles, P = 3112.32 kg;
- from the weight of the ways of riding the observation cart, P = 7926.47 kg
- from the weight of the inspection cart, P = 1587.67 kg;
- from a uniformly distributed time load q = 200 kg/m^2 accepted for the inspection trolley;
- from the weight of the two gearboxes of the inspection trolley P = 2×95=190 kg;
- from the weight of the synchronizing shaft of the inspection trolley P = 22.3 kg.

We will determine the standard linear load from the weight of the boxes for communications, railing, weight of consoles and the ways of riding the observation cart

p_1^p

$$p_1^H = \frac{1}{109.2} \cdot [9.8 \cdot (20.15377+3.11232+7.92647)] = 2.8kH/m \tag{6}$$

The estimated load is

$$p_1^p = 1.1 \cdot 2.80 = 3.08kH/m \tag{7}$$

Let's determine the total mass from the weight of the inspection cart with a uniformly distributed time load on it

$$p_c = 1.1 \cdot [1587.7 + 200 \cdot 9.7 \cdot 1.18] = 4264.59kg = 41.80kH/m \tag{8}$$

The load on one axis of the inspection trolley is

$$p = \frac{41.8}{2} = 20.9kH \text{ - on one axis} \tag{9}$$

The class of the element by load capacity is determined by the formula

$$K = \frac{k}{k_H(1+\mu_H)}, \tag{11}$$

where k – permissible time load when calculating strength, stability and endurance

k_H - the normative value of the equivalent reference load H1

$(1+\mu_H)$ - dynamic coefficient to the equivalent reference load H1

Table 1 shows the results of calculations of the classes of truss elements determined with the main and additional combination of the action of loads, a comparison of the classes of truss elements with the classes of circulating and prospective rolling stock is given [5-6].

Table 1. Classes of truss elements of the channel superstructure L_p =109.2 m with the main and additional combination of the action of loads

Farm element cipher	Farm element class						
	the main combination of the action of loads				additional combination of load actions		
	endurance		stability	endurance	endurance		stability
	by section	at the junction			by section	at the junction	
1	2	3	4	5	6	7	8
HO-1	15.70	13.95	-	12.09	14.03	12.37	-
H2-3	14.59	11.92	-	12.33	13.52	10.84	-
H3-4	15.55	12.75	-	12.98	14.22	11.42	-
H4-5	16.10	13.20	-	13.44	14.53	11.62	-
H5-6	17.12	17.13	-	14.14	15.40	15.41	-
H6-7	17.84	16.22	-	14.70	16.24	14.61	-
H7-8	17.84	17.34	-	14.70	16.22	15.71	-
B1'-2'	13.14	19.42	15.00	-	13.04	19.66	15.00
B3'-4'	13.92	13.56	15.78	-	13.30	12.93	15.15
B4'-5'	17.42	11.00	15.83	-	17.24	10.49	16.66
B7'-8'	17.61	12.63	16.00	-	17.26	12.01	15.57
P0-1'	15.30	11.41	16.75	-	-	-	-
P1'-2	13.96	12.95	252.46	11.28	-	-	-
P2-3"	12.53	13.28	13.53	14.73	-	-	-
P3"-4'	13.15	14.25	15.66	15.41	-	-	-
P4'-5"	12.95	11.96	26.68	7.53	-	-	-
P5"-6	14.22	13.64	20.30	7.30	-	-	-
P6-7"	19.61	17.82	20.25	11.07	-	-	-
P7"-8'	21.02	19.54	28.58	9.50	-	-	-
C1'-1	9.50	9.50	15.97	8.22	-	-	-
C2"-2	23.97	23.97	-	19.50	-	-	-
C4"-4	16.37	16.37	-	15.06	-	-	-
C6"-6	14.42	14.42	-	13.77	-	-	-
C8"-8	39.50	39.50	-	31.63	-	-	-
S7"-8"	13.53	13.53	-	9.06	-	-	-

*This calculation was carried out by Doctor of Technical Sciences, Professor Zhalairov A.K.

Classification of metal superstructure with through main trusses

Number of the superstructure (along the course of km) – 1.

Ride level - Lower.

Number of tracks on the superstructure 1.

Type of bridge fabric on crossbars with two separate sidewalks with a flooring of boards

The year of manufacture of the superstructure - 1927.

Design standards, year of publication - 1925.

Design load - 1925.

The material of the superstructure is Different...

The material of rivets (bolts) is Steel 2.

Type of factory connections - *Rivets.*

Type of mounting connections - *Rivets.*

Type of supporting parts - *Roller.*

The construction of the roadway – Gaps in the longitudinal beams in the panels: All 8, 8,

Estimated span_l = 109.200 m

Number of panels – 16

Length of panels d, m: extreme – 6.825

medium – 6.825

Distances between axes; main trusses B = 6,100m

longitudinal beams – 2,000 m.

Standard constant load p_i, kN/m path:

the weight of the metal of the superstructure p_1 = 61.34;

the weight of the bridge bed p_2 = 9.00.

The main design resistance of the metal R = 190.0 MPa.

The location of the bridge - 3. The elevation of the bottom of the superstructure above the level of the interline is 15.0 m.

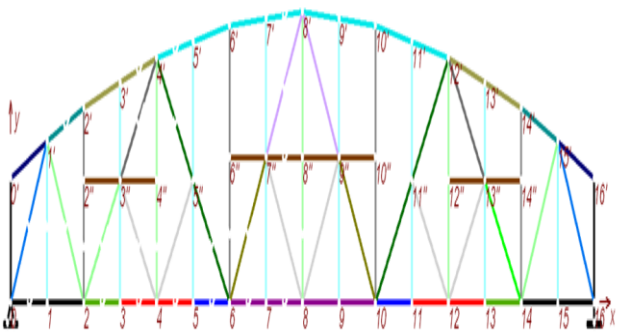

Figure 1. Farm diagram

3. Results and discussion

The load-bearing capacity of the elements of the main trusses of the superstructure L_p = 109.2 m and their attachments corresponds to the carrying capacity of bridges of the II category, i.e. ensures the handling of trains with wagons having a linear load of up to 105 kN/m of track (10.5 ts/m of track) with a load from the axis of locomotives and wagons on rails up to 270 kN (27 vehicles), and also allows the passage of conveyors with a load capacity of up to 300 tons at a speed of no more than 40 km/h and at a speed of no more than 25 km/h with a load capacity of 301-500 tons.

The classes of longitudinal beams in strength and endurance, the class of transverse beams in endurance correspond to the load capacity of bridges of category V.

The low endurance classes of the roadway indicate the accumulation of fatigue damage in these elements when exposed to trains. These accumulated fatigue damages lead to fatigue destruction of the rivets attaching the longitudinal beams to the transverse ones and the formation of gouges in the upper belts of the longitudinal beams.

Based on the results of previously performed work, classes of longitudinal and transverse beams were determined according to their design dimensions without considering damage. Tables 1 and 2 show the classes of these beams by normal stresses, by the stability of the compressed girder belt and by endurance.

Here, Tables 1 and 2 show classes of circulating and prospective rolling stock. A comparison of the classes of longitudinal and transverse beams with the classes of rolling stock indicates that in the absence of damage in these beams, the rolling stock could be passed without a speed limit.

Due to the fact that the class of the longitudinal beam, considering the damage, turned out to be low, a speed limit of 25 km/h was introduced on the bridge at one time.

In this regard, when developing a working draft, in order to determine the optimal repair option, several options for strengthening the longitudinal beam of the roadway were considered. Four options for strengthening the longitudinal beam were considered. To determine the class of the reinforced longitudinal beam, the permissible temporary load was initially determined for all four variants of its reinforcement.

The permissible temporary load kN/m on the longitudinal beam when calculating the impact of constant loads and temporary vertical load from rolling stock with the main combination when calculating normal stresses was determined by the formula

$$k_n = \frac{1}{e_k \cdot n_k \cdot \Omega_k}[\chi_2 \cdot m \cdot R \cdot c \cdot w_0 - e_p \cdot p \cdot \Omega_p] \qquad (12)$$

where e_k - the proportion of vertical load from rolling stock per beam, e_k =0.5;

n_k - reliability coefficient to vertical load from rolling stock, for l_p=6,825m n_k =1,143;

n_k - reliability coefficient to vertical load from rolling stock, for l_p=6,825m n_k =1,143;

Ω_k, Ω_p - areas of the bending moment influence line loaded from rolling stock and constant loads, Ω_k = Ω_p = 5.81m^2;

χ_2 - the dimension coefficient equal to 0.001 when calculated in the SI system and 0.01 when calculated in the GHS system is assumed χ_2 = 0.001;

m - coefficient of working conditions, m = 1.0;

R - the main design resistance of the metal, R =190 *MPa*;

c – correction factor to the calculated moment of resistance, c=1,1;

w_0 - the calculated moment of resistance of the cross section of the beam, cm^3;

e_p - the proportion of constant load per beam, e_p = 0.5;

p - total calculated intensity of constant loads, *кH/m*,

it was determined by the formula $p_p = \sum n_{pi} \cdot p_i$.

Determination of fatigue life was carried out according to the methodology developed in the NIL bridge structures of NIIZHTA. It is designed to assess the fatigue life of the elements of the main trusses of riveted superstructures working on tension (lower belts, braces and suspensions). Damage to an element or its branch by a fatigue crack is taken as a failure criterion [7-8].

The methodology is based on a comparison of the functions of the durability distribution of the elements of the superstructures of real bridges and laboratory samples, which were tested in NIIZHt. According to the test data, the equations of fatigue curves and the characteristics of the durability spread of the samples were obtained, allowing calculations of accumulated fatigue damage and comparing their results with cases of failures of real elements of superstructures.

According to the adopted methodology, the operability of the superstructure element is estimated by the calculated value of the residual fatigue life No:

$$N_o = N_p - N_э \qquad (13)$$

where N_p - is the calculated resource of the element (operating time to failure);

$N_э$ - the running time of the element at this point in time.

The impact of a conditional reference train is taken as a unit of measurement of resource and operating time. The composition of the locomotive VL-80 and four-axle full-load wagons with an axle load of 21 tc was adopted as the reference. The calculation of the value of Np in the number of these trains is carried out according to a special computer program, considering the features of the cyclic operation of this element (a set of stress cycles in the element from the passage of the reference train), the calculated characteristics of fatigue resistance and the permissible probability of failure of p.

The set of stress cycles under the reference train is obtained by «rolling» the train circuit along the line of influence of stresses in the element under consideration. To reflect the individual characteristics of its operation, the value of the constructive correction k is introduced to the levels of design stresses in the element. The value of k correlates the theoretical stress values obtained from the calculation of the superstructure according to a flat hinge scheme with their real values, which are usually found by the test results. Due to the lack of information about the tests of these superstructures, the values of structural corrections were taken according to the average values of k obtained from the tests of the corresponding elements of other superstructures /13/: for the elements of the lower belts - 0.69, for braces - 0.82, for suspensions - 0.79.

Durability calculations are based on the application of the hypothesis of linear summation of accumulated fatigue damage:

$$\nu = \sum(n_i/N_i) \qquad (14)$$

where ν - is the relative accumulated fatigue damage;

n_i - number of loading cycles of the i-th level;

N_i - cyclic durability at the i-th level of loading.

To determine the value of Ni, the following equation of the fatigue curve is used

$$lgN = A*B^{(1/1-\rho)}*(\sigma'_в/\sigma_{max} -1)^c \qquad (15)$$

where N - is the number of cycles before destruction;

ρ - the coefficient of asymmetry of the loading cycle;

σ_{max} - maximum voltage in the cycle;

$\sigma'_в$ - conditional strength limit $\sigma'_в$= 400 Mpa;

A,B,C - coefficients.

The resulting values of the calculated resource correspond to the probability of failure 0.02. This means that when the N_p value is exhausted, failure can occur with a probability of p = 0.02, i.e. out of a hundred elements operating under the same conditions, two will collapse.

The results of determining the values of the calculated resource of the predominantly stretched elements of the main trusses of the superstructures Lp = 109.2 m are shown in Table 2.

The available experience in calculating the residual fatigue life indicates that in the absolute majority of cases, elements with a calculated resource value of Np more than 4...5 million reference trains have virtually unlimited (at least in the near 25...30 years) reserves of fatigue durability. Based on these considerations, the elements of the lower belt and the C4'-4 suspension were excluded from further consideration. For the remaining elements, further calculations of operating time and residual resource were made.

The calculations of the operating time were carried out considering the number of freight and passenger trains that have passed, the structure of freight traffic, information about the types of locomotives and the loading of wagons for the entire history of the bridge operation. This information was obtained from statistical reporting forms, as well as from the data of the road department (for the period of the survey). Calculations were carried out for schemes of mixed trains based on a mathematical model of mobile units.

For the convenience of describing load conditions, the concept of the operating time coefficient Kn is introduced, which is found as the ratio of the calculated accumulated damage from the train in question to the accumulated damage from the reference one.

Table 2. Results of calculation of the calculated resource for fatigue of the elements of the main trusses of the channel superstructures of the bridge

Element designation	Characteristics of the rivet joint	Working area of the section, cm^2	Voltage from the calculated reference train, MPa		Estimated resource Nr, thousand conditional reference. trains
			maximum	minimum	
H 0-2	double shear	311.72	41.9	19.2	10120
H 2-4	double shear	613.60	48.2	22.2	6210
H 4-6	double shear	660.80	51.6	23.9	4870
H 6-8	double shear	767.70	45.5	21.0	7650
P 1'-2	single shear	199.60	60.7	26.2	1742
P 4'-5"	single shear	147.48	39.8	2.1	2785
P 5"-6	single shear	147.48	34.6	12.1	3190
C 4'-4	single shear	128.50	39.6	14.3	5741
Suspensions	single shear	75.80	38.5	7.2	2143

For a mixed train traffic, a statistically representative set of trains corresponding to the specified parameters of freight traffic is characterized by an average value of the operating time coefficient.

Table 3. Results of calculation of fatigue life for elements of superstructures with limited durability reserves

Period of operation	The number of trains that have passed, Ni is odd. direction + even direction	Operating time of PS elements, thousand fl. Trains			
		Raskos P1'-2	Raskos P4'-5"	Raskos P5"-6	Suspensions C1-1' C3-3" C5-5" C7-7"

Before 1940	58.4 + 58.8	9.9 + 16.5	15.1 + 18.8	12.3 + 16.5	90.5 + 98.3
1940 – 1955 y.	110.0 + 109.6	38.5 +42.9	58.3 + 51.5	51.7 + 46.0	101.2+ 101.9
1956 – 1975 y.	208.1+223.1	77.0 + 87.0	108.2 + 104.8	95.7 + 93.7	153.9 + 194.1
1976 – 1990 y.	167.8 + 174.3	125.8 + 130.5	156.2 + 165.3	154.6 + 168.8	441.8 + 539.5
1991 – 2001 y.	88.0 + 90.2	81.8 + 82.8	88.0 + 81.0	87.1 + 81.9	271.9 + 261.6
The operating time of the total $N_э$, thousand fl. Trains		693	847	808	2255
Estimated resource N_p, thousand fl. trains		1742	2785	3190	2143
Residual resource of No, thousand fl.trains		1049 (more than 50 years)	1938 (more than 50 years)	2382 (more than 50 years)	exhausted

To determine the operating time, the entire history of loading the elements of superstructures is conditionally divided into several periods, each of which is characterized by the number of trains that have passed N_i and the corresponding operating coefficients $K_{нi}$. As a result, the value of the operating time of the $N_э$ for the entire service life of the bridge is defined as the sum of

$$N_э = \sum(N_i * K_{нi}) \quad (16)$$

The results of the calculation of the residual fatigue life show that at present the elements of the main trusses of channel superstructures have significant reserves of fatigue durability and the probability of formation of fatigue cracks in them does not exceed 2%. With the existing parameters of train traffic (traffic intensity, structure of transported goods, axial loads), exhaustion of fatigue reserves will occur in the very distant future (not earlier than in 50 ... 60) years. The exception is the suspension of the main trusses (elements C1-1', C3-3", C5-5", C7-7" and symmetrical). The possible formation of fatigue cracks in the attachments of these elements to the upper nodes is due to an increased (more than 2%) probability.

The results of the calculation of operating time for crack-prone elements of superstructures are shown in Table 3.

4. Conclusions

Based on the results of the analysis of the survey of superstructures and the results of determining the load capacity of the superstructure L p = 109, 2 m, the following conclusions are made:

- the main trusses of the superstructure provide passage of the load corresponding to the load capacity of bridges of the II category;
- a large number of defects in longitudinal beams and low classes of their load capacity indicate the exhaustion of fatigue life.

To ensure trouble-free passage of loads corresponding to the II category of load capacity, it is necessary to replace the longitudinal beams of the carriageway, strengthen the lower belts of the cross beams or replace the cross beams.

According to the results of the calculation of the load capacity of the carriageway of the superstructure, the class of longitudinal beams under normal stresses correspond to the load capacity of bridges of category V. To ensure the passage of trains with wagons having a running load of up to 7.5 ts/m with an axle load of up to 26 ts, it is necessary to limit the speed to 60 km/h.

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Темір жол көпірінің негізгі металлды фермасының жүк көтергіштігін есептеу

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Андатпа. Бұл жұмыста қозғалыстағы бірліктердің математикалық моделіне негізделген аралас пойыздар схемалары үшін есептеулер берілген.Жұмыс уақытын есептеу жүк және жолаушылар пойыздарының өтетін санын, жүк тасымалының құрылымын, локомотивтердің түрлері және көпірдің бүкіл пайдалану тарихында вагондарды тиеу туралы мәліметтерді ескере отырып жүргізілді.Әдістеме нақты көпірлердің қондырмалары элементтерінің төзімділігін бөлу функцияларын салыстыруға және сынақтары НИИЖТ-да жүргізілген зертханалық үлгілерге негізделген. Сынақ деректері негізінде тозу қисықтарының теңдеулері және үлгілердің беріктігінің тозу сипаттамалары алынды, бұл жинақталған тозулар зақымдануын есептеуге және олардың нәтижелерін аралық конструкциялардың нақты элементтерінің істен шығу жағдайларымен салыстыруға мүмкіндік берді. Теміржол көпірінің негізгі металл фермасының жүк көтергіштігін есептеудің негізгі мақсаты жүктердің негізгі және қосымша комбинациясымен анықталған ферма элементтерінің кластарын есептеу нәтижелері болып табылады.

Негізгі сөздер: *математикалық модельдер, сенәмдәләк, жүктеме, көпірлік төсеме, аралық құрылым, жүккөтергіштік, төзімділік, тұрақтылық, элемент класы, теміржол көпірі*.

Расчет несущей способности основной металлической фермы железнодорожного моста

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Аннотация. В данной статье представлены расчеты для схем смешанных поездов, основанные на математической модели движущихся единиц. Расчеты времени работы проводились с учетом количества проходящих грузовых и пассажирских поездов, структуры грузопотока, информации о типах локомотивов и загрузке вагонов за всю историю эксплуатации моста. Методика основана на сравнении функций распределения долговечности элементов пролетных строений реальных мостов и лабораторных образцов, испытания которых проводились в НИИЖТ. На основе данных испытаний были получены уравнения кривых усталости и характеристики разброса долговечности образцов, которые позволили рассчитать накопленные усталостные повреждения и сравнить их результаты со случаями разрушения реальных элементов пролетных строений. Основной целью расчета несущей способности основной металлической фермы железнодорожного моста являются результаты расчетов классов.

Ключевые слова: *математические модели, надежность, нагрузка, настил моста, пролетное строение, несущая способность, долговечность, устойчивость, класс элемента, железнодорожный мост*.

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