SEISMIC SAFETY OF BRIDGE CRANE STEEL STRUCTURES OPERATING IN NPP

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Abstract. The seismic stability assessment of systems and equipment operating in Nuclear Power Plants (NPP) is a very important preventive precaution for ensuring their emergency safety. The actuality of this case study arises extremely after Fukushima NPP disaster, caused by the earthquake on 11 March 2011. Bridge cranes are a significant and integral part of the technological equipment, which affects the modern NPPs' safety. The main aim of the present study is to propose an analytic-simulation approach for analysis of bridge crane steel structures dynamic behavior during earthquake impact as to evaluate mostly the sliding ability of crane traveling wheels on rail roads. Loads and harmful effects acting on the crane bridge steel structure caused by the sliding phenomenon are defined and analysed.

Keywords: bridge crane, steel structure, seismic impact, sliding, safety

1. Introduction

The present days' break downs and accidents in NPPs caused by earthquakes become more frequent. Particularly after Fukushima NPP (caused by earthquake in Japan on 11 March 2011) engineers and researchers have paid attention to seeking of effective approaches and methods for assessment of NPP systems and equipment behavior. The seismic stability proof of systems and equipment operating in NPPs is a very important preventive precaution for ensuring their emergency safety. In this connection a particular place is taken by the bridge cranes, which are a significant and integral part of the technological equipment, which affects the modern NPPs' safety.

During the construction of the first Bulgarian NPP in 1970, the site located near Kozloduy town was evaluated under VI-th degree of seismic intensity by MSK-64 scale [3] and in a view of that the energy blocks and their equipment inside were not ensured against seismic impact. During the Vrancha earthquake on 4 March 1977 was measured VII-th degree of seismic intensity by MSK-64 scale at Kozloduy NPP site. According to the operative norms [7] for VII-th degree earthquakes conforms maximum acceleration for free field $a_g \approx 0.1$ m/s^2. After the failure in Chernobil's NPP in 1986, International Atomic Energy Agency (IAEA), increase again anti-seismic requirements for all the European NPPs. There begins the new seismic re-evaluation and new seismic qualification for energy blocks and equipment inside of NPPs.

In view of high degree of hazard for NPPs, have been developed a special group of standards [1 and 2] by IAEA for seismic qualification, not only for NPPs at all, but also for their equipment. Bridge cranes are one of the most numerous group material handling equipments, which operate in NPPs. Question with seismic qualification of hoisting machines in accordance with Russian norms and standards is not exactly a new question, but it is regulated in many normative documents [4, 5, 6, 7, 8]. Even so there is not known a research or normative document, in which to be analyzed the specific character of contact between crane bridge traveling wheel and rail. Likewise complicated dynamic behavior of structure in case of sliding between crane traveling wheel and rail during earthquake.

The main aim of the present development is to propose an analytic-simulation approach for analysis of bridge crane steel structures dynamic behavior during earthquake impact as to evaluate mostly the sliding ability of crane traveling wheels on rail roads. Researches have to be pointed to analysis of loads acting on the crane bridge, caused by sliding effect, and also to specific engineering solutions for protection the crane structure against non-allowable loads.

2. Normative requirements to steel structures of bridge cranes operating in NPPs

In accordance with the requirements of the design norms [7], one of the recommended load combinations for study of bridge crane steel structures in case of earthquake includes loads of normal operation and loads caused by seismic impact.

According to the requirements for construction and safety operation of hoisting cranes in sites of

NPP [8], there are admissible dynamic loads of "normal operation" and "non-normal operation" which are not included in operational loads of crane.

In accordance with [8], the construction and the calculation of cranes operating in NPPs must be performed, so that the strength requirements should not be contravened, in case of crane bridge and crane trolley sliding in longitudinal and transverse direction of the crane railroad direction during seismic impact (figure 1).

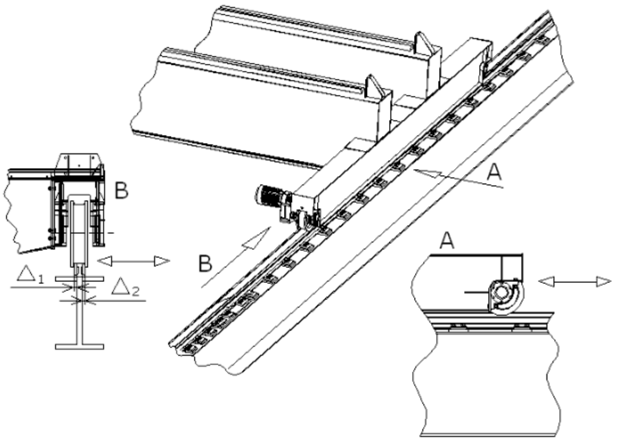

Figure 1. Sliding of a crane bridge longitudinal and transverse the crane railroad direction

On figure 1 with Δ_1 and Δ_2 are designated the corresponding distances between the rail top and the wheel flanges, for which is in force the equality:

$$\Delta_1 + \Delta_2 = B_{XK} - C_P \tag{1}$$

where B_{XK} is the distance between wheel flanges; C_P – width of rail top.

According to [10], recommended values of B_{XK} and C_P are selected depending on the traveling wheel diameter D_{XK}, as for one diameter of travelling wheel are possible several values of distance between wheel flanges. The final selection of B_{XK} has to be performed, as numerous additional factors are taken into account, such as crane span (distance between rails), technological and functional requirements etc.

Sliding effect of crane bridge on the rail is mostly a result of:

- seismic force, which is acting on crane bridge in horizontal plane, and
- friction forces between travelling wheels and rail.

Inertia seismic force, acting on a pointed mass of bridge crane construction (figure 1), in each of the three directions X, Y and Z, is calculated by the known equation [11]:

$$F_C(t,x) = m(x)\cdot a(t) \tag{2}$$

where $F_C(t, x)$ is the inertia seismic force, depending on time t and coordinate x, which is a linear parameter, indicating the location of crane trolley on the bridge; $m(x)$ - lumped mass to a point of crane structure (in most cases for such point is selected the geometric centre of crane bridge or geometric centre of front beam); $a(t)$ - seismic acceleration of pointed mass in the corresponding direction.

According to the shock theory [13], the seismic force can be calculated by the equation:

$$F_C = (1+k)\frac{m_3\cdot m_4}{m_3+m_4}\frac{\dot{y}_{r.MAX}}{\tau}, \tag{3}$$

where τ is durability of the impact - from practice it is known, that $\tau = (10^{-3} \div 10^{-6})$ s [13]; k - coefficient of restitution by impact between two steel bodies; m_3 - the part of building construction mass, which is taken into account by impact (on the basis of methodology in [14], for the purpose of present development, it can be assumed that $m_3 \approx 1000$ kg); m_4 - the part of bridge crane mass, which is taken into account by impact; $\dot{y}_{r.MAX}$ - maximum sliding velocity of the traveling wheel on rail, which is calculated according to methodology presented in point 3.1.

3. Sliding in longitudinal direction

3.1. Mathematical model

The study of crane bridge sliding along railroad is performed with following simplifications:

- crane is without load, as it decreases the friction forces between travelling wheel and rail and causes sliding effect;
- mass of crane bridge is lumped into two masses m_1 and m_2 (figure 2), located in geometric centers of the two front beams, which are calculated by the following equations:

$$m_1(x) = \frac{m_M + m_D}{2} + \frac{[L - L_1(x)]}{L} m_{TR}, \tag{4}$$

$$m_2(x) = \frac{m_M + m_D}{2} + \frac{L_1(x)}{L} m_{TR}, \tag{5}$$

where m_M is the mass of crane bridge; m_D - mass of additional equipment installed on bridge crane (electrical cabinets, gears, monitoring and guiding equipment etc.); m_{TR} - mass of crane trolley; L_1 - distance from crane trolley to the centre line of the railroad; L - crane span.

- crane bridge reaction along the crane railroad is represented by elasticity coefficient c_1 and damping coefficient β_1.

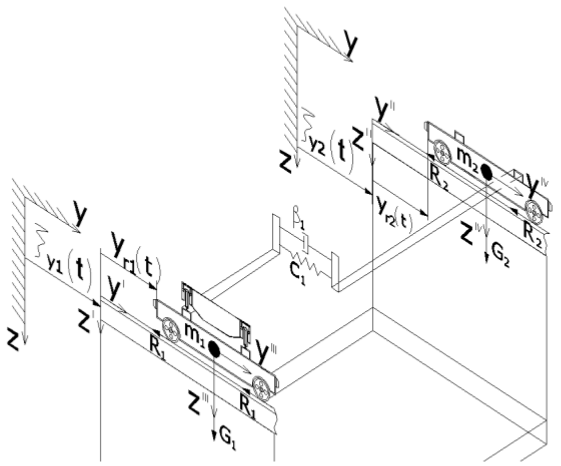

Figure 2. Mathematical model for research of crane bridge sliding along crane railroad

With help of the above defined simplifications is created mechanical-mathematical model (figure 2) for study of crane bridge sliding along crane railroad. On figure 2 are used the following denotations: $\zeta_{Y1}(t)$ and $\zeta_{Y2}(t)$ are longitudinal displacements of corresponding railroad beams regarding to the origin of immovable coordinate system XYZ; $y_{r1}(t)$ and $y_{r2}(t)$ - respectively relative movement (sliding) of front crane beam regarding to rail; G_1 and G_2 - weight, which is transferred from crane bridge to corresponding rails; R_1 and R_2 - friction forces in travelling wheels; c_1 - elasticity coefficient of crane bridge along axis Y; β_1 - damping coefficient of crane bridge along crane railroad.

Differential equations of crane bridge movement are worked out by d'Alambert principle. They represent crane sliding on corresponding rail [11]:

$$m_1 \cdot \ddot{y}_{r1} = -\beta_1 \cdot (\dot{y}_{r1} - \dot{y}_{r2}) - c_1 \cdot (y_{r1} - y_{r2}) - \\ -\mu \cdot g \cdot m_1 \cdot \mathrm{sgn}|\dot{y}_{r1}| - m_1 \cdot \ddot{\zeta}_{Y1}, \tag{6}$$

$$m_2 \cdot \ddot{y}_{r2} = -\beta_1 \cdot (\dot{y}_{r2} - \dot{y}_{r1}) - c_1 \cdot (y_{r2} - y_{r1}) - \\ -\mu \cdot g \cdot m_2 \cdot \mathrm{sgn}|\dot{y}_{r2}| - m_2 \cdot \ddot{\zeta}_{Y2}, \tag{7}$$

where μ is friction coefficient between travelling wheel and rail.

The friction coefficient depends on numerous factors such as type of material, condition and wearing of rubbing surfaces, pressure distribution in contact area, rotation (skewing) of crane bridge in horizontal plane and particularly of type of contact between travelling wheel and rail, which question is in detail analyzed in [9].

For crane which work in roofed workshops it can be assumed [10] $\mu \approx 0.2$.

3.2. Practical example for crane sliding along crane railroad

It is analyzed a double girder crane of type KM 3001 with hoisting capacity Q = 30 t, and crane trolley located at the end of crane bridge. In this condition the more loaded front beam should have at least possible sliding in comparison to all the other cases. If seismic loads in this case are bigger than the allowable values, this would mean also potential hazard for all the other cases of crane trolley location.

As the building bearing structure is not an object of the present study, to help on simplifications of calculations, it is assumed that both of the railroad beams have the same dynamic behavior: $\zeta_{Y1}(t) = \zeta_{Y2}(t) = \zeta_Y(t)$, as it is included coefficient of amplification of free site seismic acceleration transferred to level of crane railroad. According to [15] this amplification is a product of the coefficient of loading intensity (with maximal value 2) and the coefficient, which indicates height of equipment installation (with value 3 for H = 40 m).

Maximum acceleration value of input seismic impact is selected 1g. For the present study it is generated a random seismogram and its accelerogram, presented on figure 3.

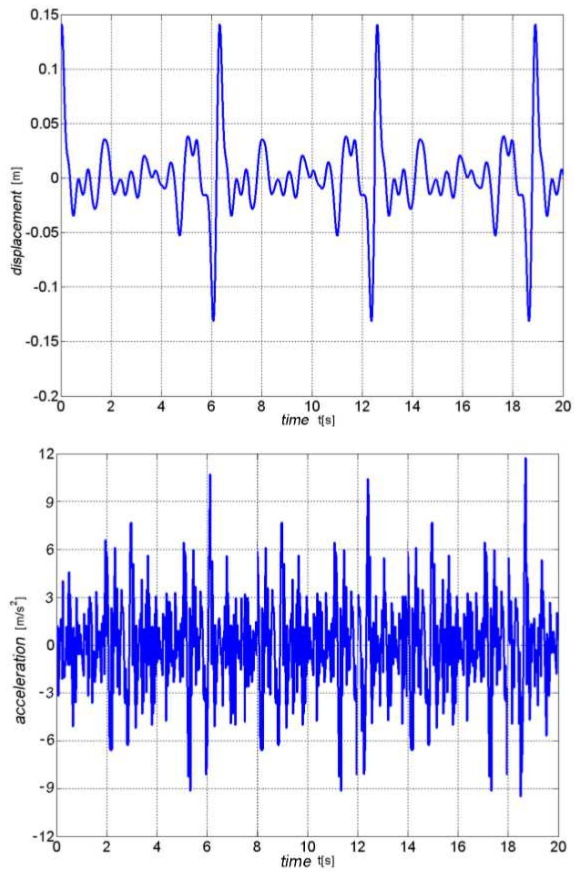

Figure 3. Input seismogram and accelerogram

After substitution of parameters' values with real data for analyzed double girder crane (m_T = 30,000 kg, L = 22.5 m, L_1 = 1.6 m, D_{XK} =0.71 m, B_{XK} = 0.11 m, C_P = 0.07 m, m_M = 22,267 kg, m_D = 10,973 kg, m_{TR} = 10,530 kg) in equations (3) and (4) are obtained the following values for lumped masses: m_1 = 26,391.8 kg and m_2 = 17,367.6 kg. In accordance with methods given in [14] are determined values of coefficient c_1 and β_1: c_1 = 61,453 N/m and β_1 = 11,223 N·s/m. Equation system (6) and (7) is solved by software product MATLAB [12], as the input seismic impact is harmonic expanded with methods given in [14].

Graphics for relative movement (sliding) of crane bridge along the crane railroad, at the side of more loaded front beam are obtained. The results are presented on figures 4, 5 and 6.

Analysis of shown graphics leads to several more important deductions. Maximal displacement of more loaded front beam is 0.38 m, in case of input seismic impact duration 20 s. This means that during earthquake event, as a result of sliding effect, there is a potential danger that the crane bridge would reach the railroad buffer support.

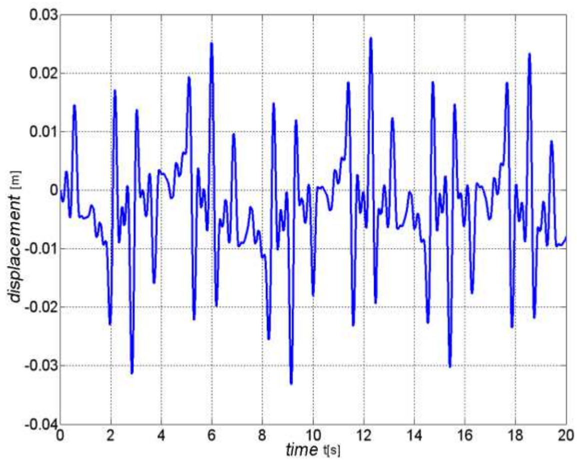

Figure 4. Relative movement (sliding)

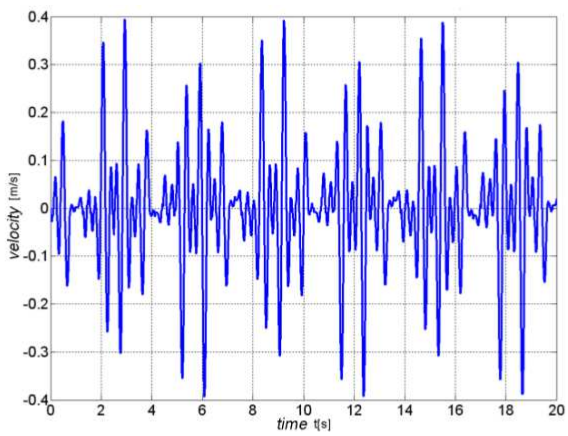

Figure 5. Relative velocity (sliding velocity)

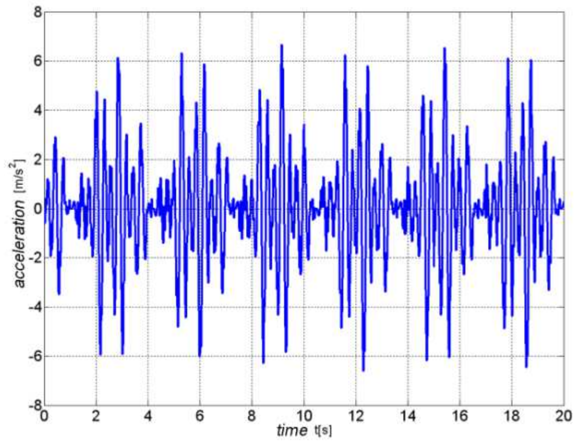

Figure 6. Relative acceleration (sliding acceleration)

Maximum value of relative velocity (sliding velocity) is 0.395 m/s, from which by equation (3) can be calculated seismic force F_C = 592.1 kN. The maximum value of sliding acceleration in more loaded front beam is 6.85 m/s^2, from which by equation (2) can be calculated the inertia seismic force, acting on the crane bridge - F_C = 180.8 kN. It is obvious, that the seismic force calculated according to "shock theory" is more than three times bigger than seismic force calculated as a normal inertia force. The calculated value of seismic force acting on more loaded front beam is significant and its ignoring is inadmissible. For the particular crane seismic force is from 42% to 138% than bridge crane own weight.

4. Sliding in direction across the railroad

4.1. Mathematical model

For study of crane bridge sliding transverse the crane railroad is created mechanical-mathematical model shown on figure 7, likewise are adopted the same simplifications as these in point 3.1.

On figure 7 are used the following denotations: c_3 is the elasticity coefficient of crane bridge along axis X; β_3 - damping coefficient of both crane main beams along axis X. The rest denotations are the same as these from figure 2.

Differential equations of crane bridge movement are worked out again by d'Alambert principle as follows (8) and (9):

$$m_1 \cdot \ddot{x}_{r1} = -\beta_3 \cdot (\dot{x}_{r1} - \dot{x}_{r2}) - c_3 \cdot (x_{r1} - x_{r2}) - \\ -\mu \cdot g \cdot m_1 \cdot \operatorname{sgn}|\dot{x}_{r1}| - m_1 \cdot \ddot{\zeta}_{X1}, \quad (8)$$

$$m_2 \cdot \ddot{x}_{r2} = -\beta_3 \cdot (\dot{x}_{r2} - \dot{x}_{r1}) - c_3 \cdot (x_{r2} - x_{r1}) - \\ -\mu \cdot g \cdot m_2 \cdot \operatorname{sgn}|\dot{x}_{r2}| - m_2 \cdot \ddot{\zeta}_{X2}. \quad (9)$$

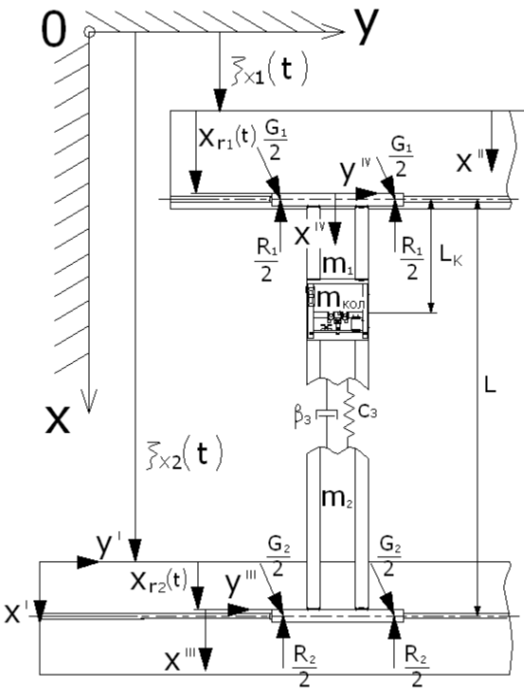

Figure 7. Mathematical model for research of crane bridge sliding across crane railroad

4.2. Practical example for crane sliding transverse the crane railroad

As an input seismic impact is used again in the seismogram from figure 3. There is assumed similar dynamic behavior of the both railroad beams, and also crane trolley located at the end of crane bridge, because of considerations given in point 3.2. In accordance with methods given in [14] are determined values of coefficient c_3 and β_3: c_3 = 29,374,737 N/m; β_3 = 25,836 N·s/m.

It is made a substitution in equations (8) and (9) with real data parameters for analyzed double girder crane type KM 3001, as in point 3.2 and the equation system is solved by software product MATLAB [12].

On figures 8, 9 and 10 are shown graphics for relative movement (sliding) of crane bridge transverse the crane railroad, at the side of more loaded front beam.

From the presented graphics it is obvious that, the maximum displacement of more loaded front beam is 0.059 m, which is much bigger than maximum distance (gap) between travelling wheel flange and rail top, calculated with substitution of the particular numerical data in equation (1): $\Delta_1 + \Delta_2 = 0.04$ m.

From the graphic shown on figure 10, can be determined, that acceleration with value 9.1 m/s^2 is reached in time 1.21 s since the earthquake event beginning, for which time after comparison with graphic shown on figure 8, is obvious that maximum displacement is 0.043 m. This displacement value shows that there is a potential hazard to have a contact between wheel flange and rail top. Maximum value of relative velocity in time 1.21 s is 0.43 m/s, from which in accordance with "shock theory" (by equation (3)) is obtained value of seismic force F_C = 644.48 kN.

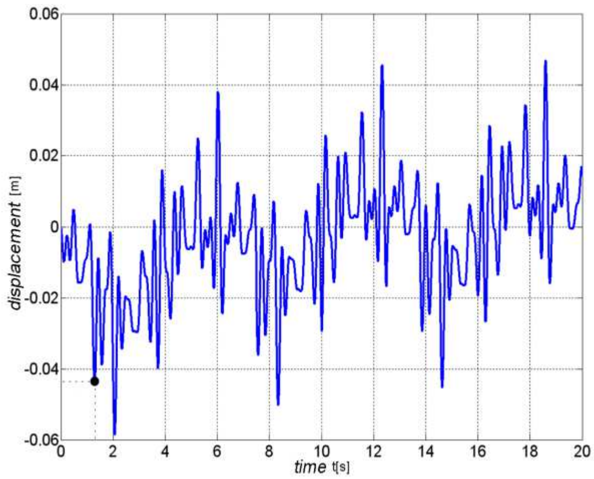

Figure 8. Relative movement (sliding)

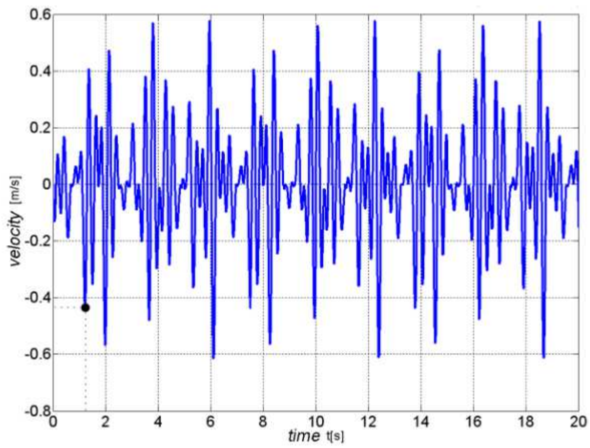

Figure 9. Relative velocity (sliding velocity)

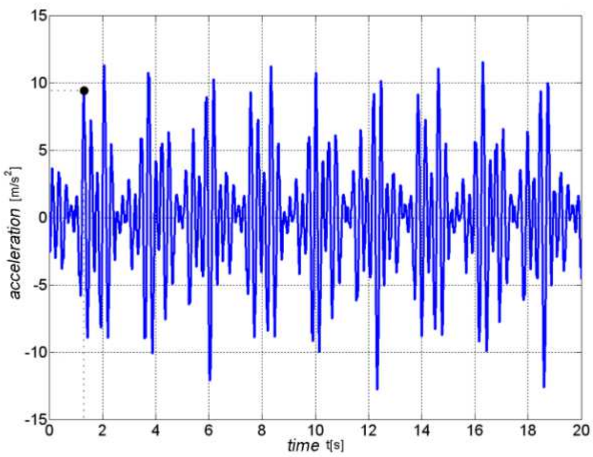

Figure 10. Relative acceleration (sliding acceleration)

For the calculated acceleration value, which is considered that can be realized in practice – 9.1 m/s^2, can be calculated inertia seismic force by equation (2), which for the particular example is F_C = 240.18 kN.

The obtained seismic force, which is acting on the wheel flange, again reaches significant values regarding to bridge crane own weight (from 56% to 149%). This force has a considerable value and by its action causes significance shear (tangential) stresses in wheel flanges, which must be taken into accounted by mechanical engineers. It can be mentioned, that seismic force, calculated in accordance with "shock theory", is more than 2.5 times bigger than this calculated in equation (2).

5. Constructive measures for safety of bridge crane steel structure

5.1. Safety in direction transverse the crane railroad

Constructive solutions for ensuring the safety of bridge crane steel structure in direction transverse the crane railroad are connected with averting an eventual crane slipping out the railroad. These decisions can be considered in two levels: first level – protection ensured by traveling wheels' flanges; second level – protection ensured by additional mounted safeguards.

The additional mounted safeguards can be implemented as shown on figure 11.

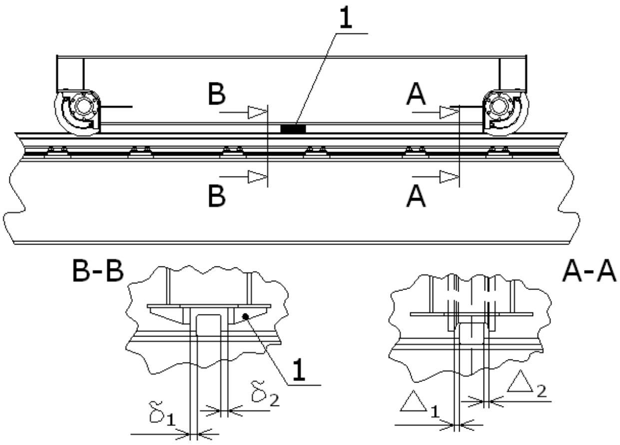

Figure 11. Safeguards against crane slipping out the rail in direction transverse the crane railroad

On figure 11 with position 1 is designated the permanent engaged safeguard against crane slipping out of the rail, for which must be observed the following constructive requirement:

$$\delta_1 + \delta_2 > \Delta_1 + \Delta_2 , \qquad (10)$$

where δ_1 and δ_2 are distances (gaps) between the safeguard and rail; Δ_1 and Δ_2 - distances (gaps) between wheel flanges and rail. By determining the values of distances (gaps) δ_1 and δ_2, the main requirement, which must be observed, is to decrease the value of dynamic impact forces, calculated by method given in point 4, to allowable levels.

By strength calculations of travelling wheel flanges and additional safeguards, the main strength check must be made with seismic loads, calculated in accordance with point 4.

5.2. Safety in direction along the crane railroad

For averting crane bridge sliding along the crane railroad is provided temporary engaged locking mechanisms, from type shown on figure 12, which are engaged only in crane parking position or in exact operation points located along the crane railroad.

On figure 12 are used the following denotations: position 1 – driving mechanism; position 2 – locking bar; position 3 – frame, welded to the crane steel structure; position 4 – frame, welded to the crane railroad; γ_1 and γ_2 - distances between the locking bar and the frame, welded to the crane railroad in transverse direction; γ_3 and γ_4 - distances between the locking bar and the frame in longitudinal direction.

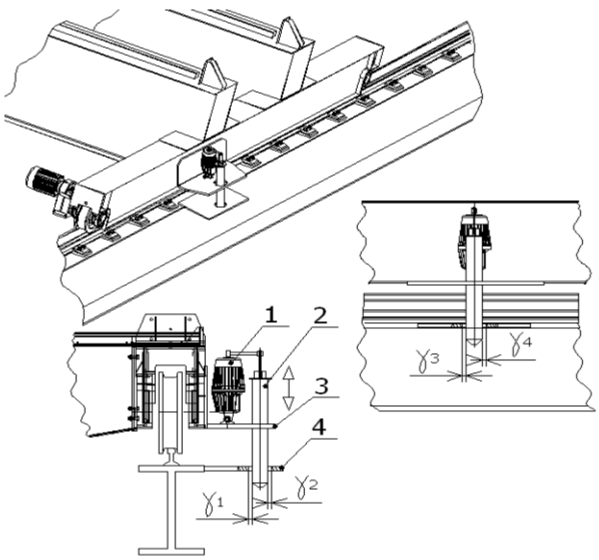

Figure 12. Locking mechanisms of crane bridge

Such type of locking mechanism restrict mostly crane bridge sliding along the crane railroad, but also transverse the crane railroad, in crane non-working condition.

By determining the values of distances γ_3 and γ_4 must be observed not only the requirement for necessary braking distance for crane bridge positioning along the railroad, but also the requirement for decreasing the values of dynamic forces, calculated by the approach given in point 3, to allowable levels. For γ_1 and γ_2 is mostly necessary to be observed the requirement for decreasing the values of dynamic forces, calculated by approach given in point 4, to allowable levels.

The main strength check, which is necessary to be performed by designing the locking mechanisms, should be implemented with seismic loads, calculated in accordance with point 3 and 4.

6. Conclusions

On the basis of the performed study, analysis and generalizations of the obtained results the following more important conclusions can be emphasized:

6.1. An analytic-simulation approach for study of bridge crane steel structure dynamic behavior during earthquake event is proposed. The approach assists engineers to estimate the probability for sliding between bridge travelling wheels and rail in direction along and transverse the crane railroad.

6.2. Through created mechanical-mathematical models are obtained graphics for relative movement (sliding) of real double girder crane in direction transverse and along the crane railroad.

6.3. It is calculated the value of realized seismic force in accordance with two methods: by shock theory and as a normal inertia seismic force. On the basis of calculated values can be concluded that “the shock theory” gives much bigger values (from three to five times).

6.4. Loads and harmful effects acting on the crane bridge steel structure caused by the sliding phenomenon are defined and analyzed

6.5. Specific engineering solutions for protection crane structures against non-allowable loads in direction transverse and along the crane railroad are proposed.

In conclusion, it should be noticed that the present study can be used as an effective engineer tool in design practice in cases of enhanced safety requirements for bridge crane steel structures, as it is, beyond all questions, bridge crane operating in NPPs.

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