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METHODS TO ASSESS BEARING CAPACITY OF CORRUGATED METAL STRUCTURES

Josef Luchko; Vitalii Kovalchuk

Dnipropetrovsk National University of Railway Transport Named After Academician V. Lazaryan Lviv branch, Lviv, Ukraine

Summary. *The aim of the research is to analyze the methods of calculating and evaluation of the bearing capacity of corrugated metal structures (the CMS) in interaction with soil backfill as a result of stress from rolling stock or vehicles. To prove the applicability of these methods for the calculation of pipes of different diameters and different structural forms. The methods of calculating the deflected mode of corrugated metal pipes of small diameter (up to 3 m.) at constant load, and calculation methods CMS of large diameter more than 6 m are analyzed. The calculated models that take into account the spatial work of structures is more correct than " flat " models and calculated scheme, and therefore the calculation of the CMS is to apply three-dimensional calculation model which model the most realistic work of corrugated metal structures. For the first time the methods of calculation and experience designing of flexible corrugated metal structures at interaction of soil backfill and under the influence of the stress from the rolling stock and road transport are analyzed. The obtained analysis of the evaluation methodologies of the bearing capacity of corrugated metal structures of different shape section can be used by engineers in bridge probationary stations Ukrainian Railroad (UR) and Ukravtodor and project organizations involved in the design and construction of the CMS.*

Key words: *corrugated metal structure, methodology, analysis, bearing capacity, bending moments, shear forces.*

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Problem setting. The problems of creating and improving methods of calculation of corrugated metal structures (CMSs) in the soil environment began to develop simultaneously with their implementation in the construction process. In the design of corrugated metal structures to designers is the task of selecting analytical valuation models bearing capacity of the CIM and in addition, given that the designers Ukraine have no experience designing corrugated metal buildings' design-soil "that this work is relevant and timely at this time.

Purpose. The purpose is to analyze and evaluate methods for calculating the bearing capacity of corrugated metal structures during the interaction with backfill dirt under loads caused by rolling stock or vehicles. In addition, the task is to prove the applicability of these methods in the calculation of pipes of different diameters and different designs.

Analysis of studies and publications. In the first stage of the assessment methods of bearing capacity of the CMSs (pipes with small diameters) "structure-ground" experimental techniques and simple calculations were used in which the CMSs were loaded by constant load, which was caused by soil pressure.

From 1900 to 1914 [1, 2] there were the following methods for calculating corrugated metal structures: Feldt’s way (1899). This method of calculating of the strength of corrugated pipes was based on the formula of tensed state of the material, which is under compressive force and bending moment:

$$\sigma = \frac{N}{F} + \frac{M}{W} \le \sigma_{гр} . \tag{1}$$

This method does not account for the movement of the horizontal section of the pipe.

In [1], which has been known since 1901 Yasevich used the hypothesis of a uniform distribution of pressure on a pipe on all sides, and received empirical formula for calculating damaging pressure on the tube:

$$p = c\sqrt{\frac{W}{dl}}\,, \tag{2}$$

where: c – empirical coefficient which equals 6; d – pipe diameter, m; l – pipe length, m.

In [1], which is known as Lyevi's way (1905) of determining external influences is done, as in Feldt's, to the top tube from one sleepers from standing on its axis of the rolling stock and pressure distribution in soil is taken at an angle of 45°. This pressure is then relatively considered as evenly distributed by the pipe surface. Maximum tension according to Levi is calculated using the formula:

$$\sigma = \frac{q\,r}{\omega} \le \sigma_{гр}\,, \tag{3}$$

where r – pipe radius; q – intensity of the relatively distributed tension; ω – sectional area of the tube strips ring.

Using these methods to practically calculate corrugated metal structures, we concluded the significant differences in calculated results [1] obtained by the above described methods with experimental data and the need for further development of methods for calculating the corrugated metal pipes.

Print elastic soil [3] has been considered on the second stage of the calculation methods of CMSs since the mid 30s. There was used the model of loose body for limit balance state of backfill for vertical σ_z and horizontal σ_x tensions in the test soil array at depth z from the surface and the following formula were received to assess tensed condition of corrugated structures:

$$\sigma_z = \gamma z;\ \ \sigma_x = \xi_a \gamma z, \tag{4}$$

where γ – unit weight of soil; $\xi_a = tg^2\left(45^\circ - \frac{\varphi}{2}\right)$ – ratio of active lateral pressure of soil.

In [1, 4, 5] it is concluded that the pressure is not equal to the weight of the soil above the pipe, and it can be considered by the introduction of an appropriate concentration factor of soil pressure and formula (4) is the following

$$\sigma_z = C\gamma z\,. \tag{5}$$

In [1] A.A. Herzog notes that vertical pressure on the corrugated metal pipe with a diameter from 0.61 to 1.22 m at the filling height of 10.5 m over the top of the structure is more than 50% by weight of the column of soil above the building, and for concrete pipes in the same conditions it reaches up to 150%.

In [5] it is found that for rigid structures with insignificant transverse deformations the coefficient of the greater concentration of vertical pressure C is higher than one and under certain conditions can reach 2, and for flexible structures, including corrugated ones, it is lower than one.

Marston-Spangler's method [3], which has been known since 1941 was proposed for the constructions with circular cross section of small diameter and further enhanced by Yaroshenko V.A. in work [6]. It is based on the assumption that the upper and lower structures are exposed to a uniform vertical pressure of backfill (soil) and lateral surfaces are exposed to horizontal pressure filling, which varies according to parabolic law. The main factor of the is the structure deformation is bending moments.

In [7] M. Spengler in 1941 offers to take diameter reduction by 5% for metal pipes as the maximum value which results in a safety factor equalling to about four. Built upon numerous field experiments carried out in the 30‘s of the last century by the dean of the University of Iowa USA Anson Marston the model is often called the " of Marston-Spengler’s theory", or "formula of Iowa".

Maximum ordinate of the horizontal soil pressure is determined by the relationship:

$$p_x = \frac{\Delta x \cdot E'}{2r}, \tag{6}$$

where Δx – horizontal linear ring deformation; E' – module of horizontal soil deformation (module passive soil strain).

Horizontal tube deformation Δx is given by:

$$\Delta x = K_1 \frac{K_B P_c r^3}{E_p I_p + 0,061 E r^3}, \tag{7}$$

where K_1 – empirical factor considering the emergence of additional radial deformations caused by long-term processes in the soil backfill; K_B – coefficient of conditions of tube support on the foundation; P_c – vertical load on the ground and temporary load per unit of pipe length; r – the average radius of the tube; E_p – modulus of elasticity of pipe material; I_p –moment of pipe inertia per unit of the pipe length; E' –module of horizontal deformation of backfill soil.

Coefficient of the conditions of the pipe work on the basement K_B in the formula (7) depends on the angle of the pipe α. It varies within [0.083 ... 0.110]. In case of the foundation, which does not change its density during the operation, we take K_B = 0,1

Vertical load of the ground and temporary load per unit of the pipe length is:

$$P_c = 2r\left(p_v + p_g\right), \tag{8}$$

where p_v –temporary vertical uniformly distributed over the length of 2r of the load of traffic; p_g – vertical uniformly distributed over a length of 2r of the soil load.

The coefficient that takes into account the emergence of additional radial deformations caused by long-term processes in the soil backfill K_1 is assumed to be 1.5. In modern buildings where backfill soil is done by specially selected grain size, K_1=1,0.

Vertical compressing force in ring-section *x0y* per unit of the pipe length equals to half the load of soil and temporary load

$$N = r\left(p_v + p_g\right). \tag{9}$$

The coefficient of reliability of compressed sections is not introduced. However, the control of the value of tensions in the extreme fiber sections is performed. Moments in sections are the following:

$$M = 0,08 p r^2, \tag{10}$$

where $p=p_v+p_g$ – full-uniformly distributed vertical load on the length 2*r*.

For rigid pipes A. Marston’s pressure theory was further developed in the works of Karl Klein [3], N.M. Vinogradov [14], V.A. Yaroshenko [6] and others.

Kleyn’s method (1951) is based on the assumption that during deformation the pipe undergoes resistance of the soil, resulting in a slight decrease in tension in the pipe material. In calculation this assumption reflects in the introduction of reduction of bending moments factor [1]. As for the pressure on flexible pipes, the recommendation to this effect usually predict coefficient C equal to 1.0 [8].

As a computational model for assessing soil interaction of CMS with filling most common were Winkler's model and elastic half-plane model.

The model of Winkler's bases ($p = kw$) has been criticized because the drag coefficient k is an uncertain magnitude and can be expressed in terms of the basic characteristics of the soil – deformation module E_0 and Poisson's ratio ν_0. However, due to the simple expression of dependence $p = kw$, many authors [9, 10] are in favor of preservation of this dependence in the calculation, but at a reasonable determination of the coefficient k.

Widely used has become the calculation model of elastic half-plane. In which the connection between the environment movement at coordinates (x, y) and jet pressure p is shown in the equation:

$$w = \int_{-\infty}^{\infty}\int_{-\infty}^{\infty} p(\varsigma,\eta) K(x-\varsigma;\ y-\eta)\, d\varsigma d\eta\,. \tag{11}$$

In [11] O.Ye. Bugaeva received simple formulas for the efforts of vertical pipe with evenly distributed load in the most dangerous section of the pipe:

$$\begin{aligned} M_{00} &= 0{,}25qr^2(1-0{,}056n); \\ N_{00} &= 0{,}042qrn \end{aligned}\ , \tag{12}$$

where $n = \dfrac{1}{0{,}06416 + \left(\dfrac{EI}{kr^4}\right)}$; k – radial drag coefficient of the pipe side movement; r – середній радіус поперечного перерізу труби the average radius of the cross section of the pipe; EI – cylindrical stiffness; q – intensity of uniform vertical pressure level in the pipe arch.

The main disadvantage of this method is the assumption about the form of an elastic ring line, which does not depend on the characteristics of the soil and the structure.

To clarify the line pipe deformation R. Prevo [12] recommends to count CMS in two stages. The first phase intends to calculate the elastic ring without the ontime ground ("free ring") to vertical balanced load at the top and bottom of the pipe. In the second phase comprises estimation of the ring on the load of elastic backfill soil print.

One of the ways in which the elastic line of the construction deformation is not given, but is a result of the calculation using Metroproject method. In the calculation with this method, developed by B.P. Bodrov and B.F. Materi [13], circular axis of the underground ring is replaced by 16-andle, continuous load of the concentrated forces applied at the vertices of the polygon, and the base reaction are substituted by the resilient supports in all polygon tops, except for the top three located in unsupported area corresponding to the destination points for zero movement at the borders of the first quadrant angle ± 45°.

Proximity of the Metroproject method is to replace the continuous curve of the ring contour with the broken polygonal line as well as to replace the continuous reactions with the focused ones. In addition, the task is to mak radial elms stiffness on the same circuit pipes.

In 1936 D. Weinberg published his work [14], in which the arch is considered as flat rod of the small curvature. The author received the following differential equation (in tangential movements):

$$\frac{d^6u}{d\varphi^6} + 2\frac{d^4u}{d\varphi^4} + \mu^2\frac{d^2u}{d\varphi^2} = 0\,. \tag{13}$$

In the second paper D. Weinberg [15] examined the circular arch on elastic foundation, which not only resists radial movement of timber ω, but the tangents u as well. The corresponding equation is

$$\frac{d^6u}{d\varphi^6} + a_4\frac{d^4u}{d\varphi^4} + \mu^2\frac{d^2u}{d\varphi^2} - a_6u = 0. \qquad (14)$$

In his paper, B.G. Galerkin [16] considered axisymmetric Lame for the pipe exposed to internal pressure, temperature and embedded in the elastic environment. There was obtained relationship between the rate of deformation modulus bed k and the environment deformation module E_0:

$$k = \frac{E_0}{(1+\mu_0)r}, \qquad (15)$$

where μ_0 – Poisson's ratio of the elastic environment.

In 1952 there was L.M. Emelyanov's work [17], in which the soil around the pipe, seen as elastic environment with two characteristics. It was drafted differential equation of the sixth order as (14), but with the right side which takes into account the external load. The intensity of the load p and p_1 thus radial movement of the pipe are presented as a trigonometric series; in some rows we obtained formulas. There have been done a number of examples of practical interest. For example, if the value of the load bending moment equals to:

$$M = qr\left[\frac{9cos2\varphi}{8(9+\mu^2)} - \frac{6}{\pi}\sum_{n=1}^{\infty} a_n cosn\varphi\right], \qquad (16)$$

where $a_n = \dfrac{(n^2-1)sin0{,}5\pi n}{n(n^2-4)\left[(n^2-1)^2+\mu^2\right]}$;

$$\mu^2 = 1 + \frac{kr^4}{EI}. \qquad (17)$$

To download the ring the formula for the biggest moment is defined by the formula:

$$M = 0{,}25qr^2\xi_0, \qquad (18)$$

where
$$\xi_0 = \frac{1}{0{,}889 + 0{,}111\mu^2}. \qquad (19)$$

In [18] L.M. Emelyanov rejected the hypothesis of the "bed coefficient" and considered ground as linear- deformative elastic environment.

In [19] the results of calculation of thin-walled tube on supports based on the theory of elastic shells (V. Z. Vlasov) were compared with the basic decision of the materials resistance. At the end of the calculation arguments about the cylindrical shell partially or completely enclosed in an elastic medium were made. However, in [19, 20] tubes upper zone was not considered. According to [3] in the calculation of underground pipelines as spatial membranes should be resorted to only in cases where they have hard rings, or when they are on separate poles. Underground pipeline lies on the ground basement and works in the conditions of a plane strain and is calculated on the transverse load as a ring of the single width.

Not being able to describe many other works, we note that calculations of the flexible ring associated with the tensed print in terms of of plane problem have been done by I.A. Baslavskiy [21], G.K. Klein [22], N.N. Shaposhnik [23] and others.

In the world practice, the calculation of building structures to assess their bearing capacity is done using the method of limit states [24]. According to it, the calculation of the construction is performed for strength, endurance and crack (the first boundary condition) and excessive deformation (second boundary condition). However, for CMS in the soil there are uncertainties in the construction design scheme and determining internal forces, which complicates the consistent use of the method of boundary conditions.

A thorough review of foreign literature of underground pipelines until 1960 has been done in Robert Prevo's book. [12] The review pay attention to the lack of sufficiently accurate and scientifically based methods of calculation. Pipes calculations are based on semi-empirical formulas proposed by different authors.

Analytical method of the elasticity theory is provided in the works of Burns and Richard (1964), Hoeg (1966), Krizek (1971), Peck (1972) which is based on relationships of the problems of the plane elasticity theory. The assumption of linear elastic homogeneous, isotropic materials of structures and soil. The theory has been applied for cases of high backfill layer of the structure.

The described methods refer to the so-called traditional ("old") ones. Since the 90s of the previous century a number of contemporary research methods of compatible interaction of pliable metal structures with the soil have been developed.

The theory of ring stamping (White i Layer – 1960) provided that the pipe after filling and fairly high altitude layer of soil over the pipe structure can be modeled as a thin ring under stamping. The theory is based on the fact that uneven pressure has little effect on the schedule and axial forces (Marston, Spangler). The above fact is true for the case where the height of backfill layer structure is larger than $1/8$ of the size of the cross tube. The theory takes into account the noncircular pipe section and soil compaction during filling. The influence of bending moment is neglected.

Since *the mid 70s* of the last century methods taking into account nonlinear soil work of CMS and the soil compartment began to develop. They use powerful computing systems based on the finite element method (Kosmos, Ansys, Plaxis etc.) and soil is simulated by elastic or elastic-plastic environment. It is assumed that the accuracy of the calculations is provided by fine mesh partitioning of soil region into finite elements. However, the calculation results in many cases differ from the observed data. This is due to the fact that the calculation model of soil does not include its important properties such as increased modulus of soil depth with static and dynamic loads.

Summarizing in the 1968-1970 domestic and foreign experience of corrugated metal for building small artificial structures, especially culverts there was developed a calculation method of flexible steel pipe on marginal static equilibrium, which is laid as the basis for Technical Guidance on Design, Manufacture and Construction of Corrugated Metal Culverts (WPC 176-78) on railroads and highways [25]. This method realizes the idea of strain fracture criterion expressed by

$$\frac{dq}{df} = 0\,, \tag{20}$$

where $f = \Delta D$ – reduction of the vertical pipe diameter after acting on its vertical load q.

The hypotheses underlying the model are the following: vertical load on top of the pipe is uniformly distributed on the width D = 2r; distributed passive soil resistance on the part of the contour of the deformed pipe; in the boundary condition in the shell plastic pipe joints are formed.

The condition of the strength of the first boundary as the system "soil-structure" is to satisfy inequality

$$q = q_p\,, \tag{21}$$

where q – the estimated intensity of vertical soil pressure on the pipe of permanent and temporary loads; $q_\text{р}$ – the estimated intensity of the passive resistance of soil (pipe bearing capacity) provided static equilibrium system "structure-soil".

The estimated intensity which is bearing capacity of the pipe is defined by the formula:

$$q_p = K_{ув} \cdot q_{1.p}\,, \tag{22}$$

where $K_{ув}$ – rate of bearing capacity increase of the pipe through the spring soil passive resistance:

$$K_{ув} = 1 + \frac{12{,}1 \cdot 10^{-4}}{\sqrt{G}}\,, \tag{23}$$

where $q_{1.\text{р}}$ – estimated carrying capacity of the pipe of steel make without soil filling:

$$q_{1.p} = 0{,}032 \cdot 10^{16}\,\frac{W^2}{D^2}\,, \tag{24}$$

where W – moment of resistance section shell length (per unit of pipe length); D –pipe diameter in the corrugated pipe midline; G – generalized stiffness index of the system "construction-soil":

$$G = \frac{W}{D^2 E_{gr}}\,, \tag{25}$$

where E_{gr} – compressive deformation module backfill soil.

Limited horizontal increase in pipe diameter, corresponding static equilibrium system is determined by a similar dependence (14) obtained in the case diagrams for $q_\text{р}$ Fig. 1:

$$\Delta D' = \frac{1{,}1 q_p D^3}{0{,}96EI + 0{,}0052E_p D^3}\,, \tag{26}$$

where E – modulus of steel elasticity l; I – moment of inertia of the longitudinal section per unit of pipe length; $q_\text{р}$ – characteristic intensity value of passive resistance of the soil under the condition of static equilibrium system "structure-soil".

Calculation of pipes on the overall stability of the criterion (11) is limited to checking compressed section to the effect of the estimated compressive strength with the coefficient of reduction of load capacity to prevent buckling of pipe membrane. It is assumed that the shell is under uniformly distributed load along the pipe circuit. The value of this load calculation is assumed to be the intensity of vertical soil pressure on the pipe of permanent and temporary loads q. Stability condition is the following:

$$\frac{N}{\varphi A} \le 0{,}7R_0\,, \tag{27}$$

where A – sectional area of membrane per unit of pipe length; φ – коефіцієнт зниження несучої здатності reduction factor of bearing capacity; R_0 – estimated resistance of steel the axial forces; N – the estimated and normal to the section, centrally applied force

$$N = \frac{qD}{2}\,, \tag{28}$$

where q – as in equality (17) is estimated intensity of vertical soil pressure on the pipe of permanent and temporary loads.

In the regulations WPC 176-78 [26] is given a detailed definition of reduction factor of bearing capacity φ.

Strength of CMS calculation is performed in accordance with DBN V.2.3-14 [27] with the formula:

$$\frac{N}{A} \le R_y m\,, \tag{29}$$

where N – normal (tangential) efforts in the construction of corrugated design load, corresponding to the length λ of one corrugation, *H*; A – sectional area of a wave of corrugated pipe, cm^2; R_y - estimated resistance of steel within liquid limit adopted in accordance with the DBN V.2.3-14, Pa; m=0,9 – coefficient of working conditions.

Normal (tangential) force N in the construction of the design load, corresponding to the length λ of one corrugation is determined by:

$$N = \frac{\gamma \cdot n \times \left(h_{eq} + h + \dfrac{D}{2}\right)\cdot\lambda}{2 + \dfrac{E_0}{E}\cdot\dfrac{D}{\delta}\cdot\left(1-\nu^2\right)} + \frac{\gamma_{sh} n_1 \delta \dfrac{D}{2}\lambda}{1+\dfrac{\delta^2}{3D^2}}\,, \tag{30}$$

where γ – the proportion of backfill soil, H/m^3; n = 1,3 i n_1 = 1,1 – ratios under overload DBN V.2.3-14; h_{eq} – conditional height of the embankment is the equivalent to the temporary car load:

$$h_{eq} = \frac{q}{\gamma\cdot\left(a_0 + h\right)}\,, \tag{31}$$

where a_0 – width lane for loading NK-80 (NG -60) according to DBN V.2.3-14, m; h –distance from the top of the pavement to the top of the structures, m; q – equivalent load, according to DBN V.2.3-14 depending on the length and shape of the influence line; D –diameter of the corrugated structure, m; E_0 – backfill soil deformation modulus, Pa; E –modulus of elasticity of steel, Pa; δ –conventional sheet thickness of the round corrugated structure, which has the same running flexion stiffness as the corrugated one, eg. for corrugated structures with λ =0,164 m; ν =0,25 – Poisson's ratio of the structure material; γ_{sh} – the proportion specific weight of the material MGK, N/m^3.

The method of interaction with soil. Методика взаємодії з ґрунтом (SCI Soil-Culvert Interaction, 1983). The method was suggested by Duncan and Drawsky [29]. The method was developed based on years of research of engineering strustures conducted on observations models and performed numerical calculations by finite element method. as an Both influence of squeezing forces and bending moments in the wall construction and nonlinear stress and strain of soil have been included. It is shown that increasing soil stiffness (modulus of deformation) reduces the effect of bending moment on the stress-strain state design. Therefore, the method takes into account two phases of construction: 1) installation phase when filling has reached the top of the pipe; 2) the final phase, when the backfill has reached the projected height. A criterion operability taken to prevent the start of plastic deformation in the walls of the pipe. This is achieved by the introduction of appropriate safety factor into the calculations. The attention has been paid to backfill compaction.

Vaslestad's method (1990 p.) has been proposed for constructions of large cross-sections. It recognizes only axial forces effect it is accepted that much of the stress is taken by soil. We have investigated the carrying capacity of pipe walls in compression and deformation of the top construction during the laying and compaction of backfill soil and the effect of friction on the value of compressive force. [31] The model describes the emergence of the phenomenon of the construction fullness under the top layer of soil above the pipe.

Here are some other techniques used to estimate the bearing capacity of corrugated metal structures. One of them is OHBDC (Ontario Highway Bridge Design Code) technique. This technique was developed based on American (1992) standards of bridges design. It is based on the assumption of the dominant role of axial forces inside the tube. We have developed calculations of strength of the structures walls on compression, joints strength and mounting rigidity. The method takes into account the case of pipes with open cross-sectional, phenomenon of bursting design, and the impact of the degree of soil compaction on the module size of its deformation. Structures compliance has been included.

AASHTO (American Association of State Highway and Transportation Officials, 1996) technique. American technique developed in accordance with the design of bridges of the American union of motorways and transport workers [28]. Like OHBDC the technique ignores the influence of bending moments and only considers axial forces. Calculations of strength seams, buckling of the construction walls, and mounting rigidity are given, the possibility of plastic deformation inside the pipe is taken into account . It allows exploring the construction with the frame section type. Dynamic factors are taken into account in case of variable loads.

CHBDC (Canadian Highway Bridge Design Code) technique. This technique was developed based on the Canadian (2000) bridge design standards [30]. It is based on the assumption of the dominant role of axial forces inside the pipe. Calculations of the construction walls strength in compression, strength joints and mounting rigidity have been developed. The method takes into account the case of open pipe cross-sectional design and bursting phenomenon and influence of the degree of soil compaction on module size of its deformation. Structure compliance is included which allows to design structures of flat-top section.

Sundquist-Petterson's is one of the newest techniques (2000) [32]. It is based on the basis of the above described techniques and experience gained from the experiments on the destruction of structures using analytical approaches of elasticity and geotechnics. It has been applied in cases where the largest cross pipe cut $B \geq 2$ m and height of soil on top of the construction $H \geq 0{,}6$ m and $H/B \geq 0{,}125$ m. To assess the bearing capacity it considers axial force and bending moment, the angle of internal friction of backfill and dynamic loads from the moving vehicle. It is characterized by sufficient versatility and considers malleability of structures made of corrugated metal sheets.

Scientific novelty and practical significance.

For the first time there have been analyzed the techniques of calculation and experience of designing flexible corrugated metal structures during their interaction with soil and backfill when subjected to stress from rolling stock and road transport. It has been researched that calculated models that take into account the spatial work of structures is more correct than "flat" model and calculation schemes. As a result to calculate CMS one should use three-dimensional calculation models which most realistically model the work of corrugated metal structures.

The obtained analysis of techniques to assess the bearing capacity of corrugated metal structures of different cross-sectional shapes can be used by engineers of Bridgetesting railway stations and Ukravtodor and project organizations involved in the design and construction of the CMS.

Conclusions and prospects of research in this area. 1. Calculated models that take into account spatial construction work are more exact than "flat" models and calculation scheme, so that in the calculation of CMS it is more reliable to apply three-dimensional model.

2. As it is seen from the analysis, the use of force technique to the arch model of corrugated metal structure and finite element model without taking into account the asymmetry in the corrugated construction during loading, gives results that differ from the experimental

data, especially at high loads. Whereas, in the computer models the opportunity of asymmetric behavior was not marked, it will not affect the results of the calculation.

3. When calculating the corrugated metal structure it is necessary to pay attention not only to the behavior model of the corrugated structures, but also the right choice of backfill soil because of its heterogeneity and possible inclusion in the work, as an additional base layer.

4. The above analysis leads to the conclusion that the analysis of the action of the complex structures such as corrugated shell interacts with soil. The experimental studies research is essential part of the investigation as the construction and use of computer models without taking into account the effects which are uncovered during the experiment can lead to not always correct conclusions about the carrying capacity and corrugated structures acrion interacting with the soil.

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МЕТОДИ ОЦІНЮВАННЯ НЕСУЧОЇ ЗДАТНОСТІ МЕТАЛЕВИХ ГОФРОВАНИХ КОНСТРУКЦІЙ

Йосип Лучко; Віталій Ковальчук

Дніпропетровський національний університет залізничного транспорту імені академіка В. Лазаряна Львівська філія, Львів, Україна

Резюме. *Проаналізовано методи розрахунку та оцінювання несучої здатності металевих гофрованих конструкцій (далі – МГК) при взаємодії із грунтовою засипкою в результаті дії навантажень від рухомого складу залізниць чи автотранспорту. Обгрунтовано можливість застосування даних методів при розрахунку труб різного діаметра та різної конструктивної форми. Проаналізовано методи розрахунку напружено-деформованого стану металевих гофрованих конструкцій труб малого діаметра (до 3 м.) при постійних навантаженнях та методи розрахунку МГК великого діаметра більше 6 м. Проведено аналіз методик оцінювання несучої здатності металевих гофрованих конструкцій різної форми поперечного перетину, які можуть бути використані інженерами Мостовипробувальних станцій Укрзалізниці й Укравтодору та проектними організаціями, які займаються проектуванням і спорудженням МГК.*

Ключові слова: *металева гофрована конструкція, методики, аналіз, несуча здатність, згинальні моменти, поперечні сили.*

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