Збірник наукових праць. Галузеве машинобудування, будівництво
Academic journal. Industrial Machine Building, Civil Engineering
http://journals.nupp.edu.ua/znp
https://doi.org/10.26906/znp.2023.61.3849

UDC 624.072.2.014.2-413

The limit state of steel beams research of damaged building elements

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The article is devoted to the study of the limit state of steel I-beams under buckling with operational geometric imperfections revealed during inspections of damaged elements of reconstructed buildings. The influence of damage in the form of initial curvature in the horizontal plane on the stability and bearing capacity of structures is analyzed. The possibility of using bearing capacity resources, namely lateral bracing restraint with the help of structures attached to a steel beam, is considered. The proposed approaches are aimed at optimizing the use of materials and increasing savings by detailing the verification calculations of damaged steel beams.

Keywords: buckling, I-beam, initial curvature, operational geometric imperfections, restraint.

Дослідження граничного стану сталевих балок пошкоджених елементів будівель

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У будівлях, які експлуатуються тривалий час і зазнали пошкоджень, прокатні балки є одними із найбільш поширених елементів, що потребують обстеження для точного визначення їх технічного стану та можливості відновлення. Основним способом оцінки технічного стану конструкцій є перевірочний розрахунок, у якому здійснюється врахування дефектів і пошкоджень. Під час його виконання використовуються дані, отримані в ході натурних оглядів, контролю характеристик міцності матеріалів, уточнення навантажень і впливів. Стаття присвячена вивченню граничного стану сталевих балок при втраті стійкості з експлуатаційними геометричними недосконалостями, виявленими під час обстежень пошкоджених елементів відновлюваних будівель. Проведено аналіз впливу пошкоджень у вигляді початкових викривлень у горизонтальній площині на стійкість та несучу здатність конструкцій. Основний акцент робиться на визначенні особливостей експлуатації балок. Зокрема, розглядається можливість застосування ресурсів несучої здатності, а саме бокового розкріплення за допомогою конструкцій, приєднаних до сталевої балки. При дотриманні нормативних вимог щодо забезпечення надійного з’єднання суцільного жорсткого настилу зі стиснутим поясом стійкість балки не перевіряється. Удосконалена теоретична модель враховує і усуває присутні у попередній моделі недоліки. Запропоновані підходи спрямовані на оптимізацію використання матеріалів та збільшення економії шляхом деталізації перевірочних розрахунків сталевих балок із пошкодженнями. Висвітлені та розглянуті підходи щодо визначення допустимих викривлень сталевих двотаврових балок у горизонтальній площині можуть бути використані з достатньою точністю при відновленні пошкоджених елементів будівель. Не зважаючи на певні недоліки, якими наділені ці методи, вони достатньо прості та відповідають нормам проєктування сталевих будівельних конструкцій.

Ключові слова: втрата стійкості, двотаврова балка, початкове викривлення, експлуатаційні геометричні недосконалості, розкріплення.

Introduction

Frame buildings in general, and one-story buildings in particular, are the most common use of steel structures in the world. Until recently, hot-rolled beam profiles were predominantly used for roof purlins in industrial buildings. Although solid section purlins are heavier than lattice girders, they are much easier to manufacture and install. This type of purlin was often used in combination with steel trusses with a spacing of 6 meters. Steel beams made from hot-rolled sections can still be used instead of cold-formed thin-walled sections for roof purlins. With large roof slopes, the channel section of the purlin works well for oblique bending. Under heavy loads, which are typical for flat roofs of large shopping malls, the cross-section of the purlins can be taken from rolled I-beams (Fig. 1). Hot-rolled purlins can be used in large-scale panel solutions, where they are particularly useful for providing intermediate support for decking that is not capable of spanning large spans on its own. However, with the advent of cold-formed purlins, which are significantly lighter and less expensive, the use of hot-rolled purlins has become unusual.

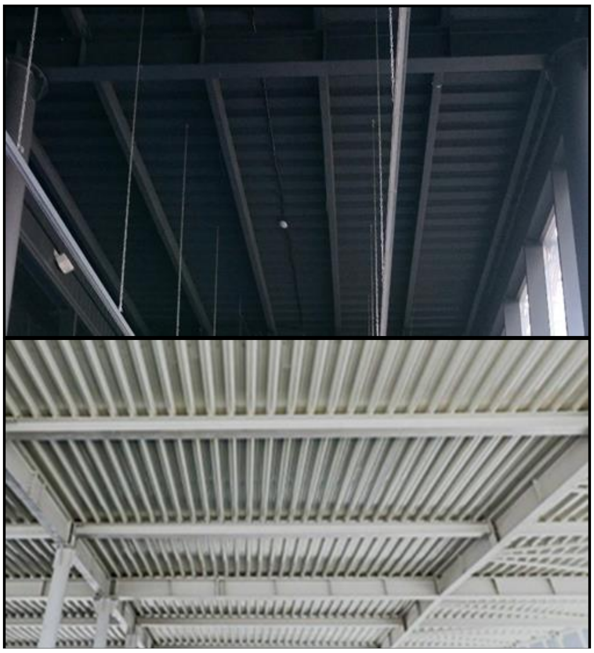

Figure 1 – View of the flat roof structures on I-beams in the transverse and longitudinal directions

Review of the latest research sources and publications

The effect of geometric imperfections (in particular, the initial curvature of the beam axis in the plane of least stiffness) is partially taken into account in the standards by introducing a working conditions factor. In compliance with the regulatory requirements for ensuring a reliable connection of a continuous rigid deck with a compressed flange, the stability of the beam is not checked. At the same time, the bracing should be designed to withstand the fictional load. In other cases, if the compression flange is insufficiently restrained, the stability of the beam should be calculated using the formula with a buckling factor. The fictional (conditional, equivalent, stabilizing) load is understood as the forces arising from geometric imperfections and deformations of the beam and acting between the structural elements. They create an dominate action on the bracing structures and a supporting action on the beam. Their determination in construction practice is often performed using approximate methods that consider the compressed beam flange separately from the rest of the section as a continuously supported compressed rod with a longitudinal force.

In recent years, much attention has been paid to the study of the stress-strain state of damaged steel beams. Numerous theoretical and experimental studies in this area consider both thin-walled steel beams without bracing and those with bracing [1, 2]. The multifactor spatial model for describing the fictional load, which is presented in the works of German scientists [3 – 5], is used under the condition of sufficient stiffness of bracing structures in the calculation of bracings and stiffening diaphragms. These papers reveal aspects of torsion and stability of beams, providing valuable theoretical conclusions. In particular, it is argued that for unbraced beams, the compressed rod model underestimates the maximum fictional load, which reduces the reliability of the structure, and in the case of sufficiently braced beams, it is shown by example that such a model leads to significant material overruns and is not economically feasible. In [6], the problems of developing a spatial model to describe the fictional loads on steel beams of asymmetric cross-section under the combined action of transverse bending and forced torsion were formulated and solved. In solving stability problems, the energy method is widely used, which is of interest and involves the automatic execution of cumbersome computer operations [7, 8]. These sources provide a snapshot and show the state of modern research on the stress-strain state of damaged steel beams, taking into account various aspects of stability and buckling.

Definition of unsolved aspects of the problem

An unresolved part of the general problem of analyzing and studying the condition of steel elements is to determine the permissible values of curvature of steel beams in the horizontal plane when restoring damaged parts of buildings. Attention should be focused on ensuring the stability and efficiency of further operation of the beams.

Problem statement

The aim of the study is to determine the effect of beam damage on their stress-strain state. As part of the study, it is planned to take into account the peculiarities of structures, adhering to the relevant regulatory documents in construction. In particular, the model for determining the load-bearing capacity of steel beams of the most common I-beam constant cross-section in length, taking into account strain and stresses resulting from damage, needs to be improved. Additionally, the causes and consequences of torsion in structures with curvature are considered. The results of this study will be useful for further improvements in the design and operation of steel structures with regard to possible and existing damage.

Basic material and results

In buildings that have been in operation for a long time and have suffered damage, rolled beams are among the most common elements that require inspection to accurately determine their technical condition and the possibility of restoration. The main method of assessing the technical condition of structures is a verification calculation that takes into account defects and damages. It uses data obtained during on-site inspections, control of material strength characteristics, and specification of loads and impacts. Overloading of the roof, in particular by snow load, leads to damage and emergency conditions (Fig. 2).

Figure 2 – Damage to roof from overloading

Structures can also be unprotected from damage and become damaged in the event of accidental explosions. For example, in the event of a fire, the steel frame of an operated building quickly heats up to the temperature of the metal's transition to a plastic state, resulting in an increase in irreversible deformations and exhaustion of the load-bearing capacity until failure occurs (Fig. 3).

Figure 3 – Appearance of metal frame after a fire and design sketch of the beam by fire effect

The fire resistance of metal structures is insignificant because metal has a high thermal conductivity. In addition, thin-walled cross-sections of structural elements are more prone to buckling when heated, which is one of their disadvantages. If damaged beam elements are reused for further operation, they develop new initial geometric imperfections that lead to torsion, which can lead to a lateral-torsional buckling.

By buckling, in addition to deflection strain, the beam receives strain of curvature and rotation of the cross-section. The spatial model of the beam strain is shown in Fig. 4.

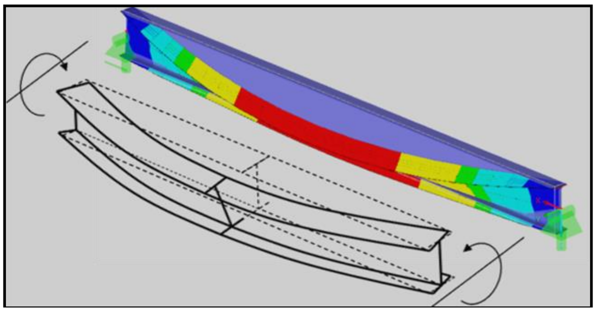

Figure 4a – Beam strain during buckling

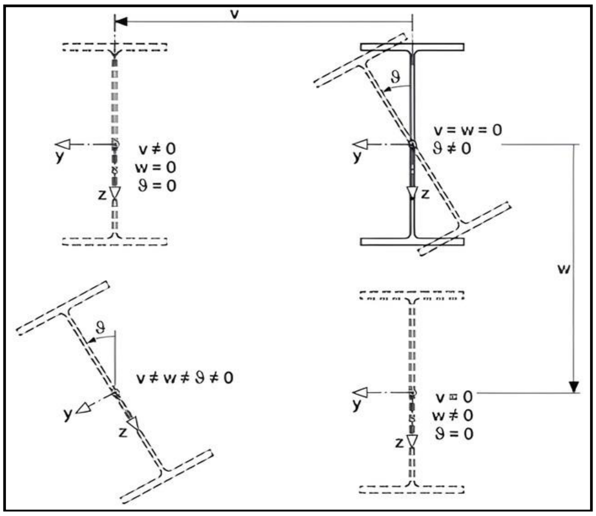

Figure 4b – Actual beam performance

To find the permissible value of the initial curvature of damaged beams in the horizontal plane, it is possible to use the formulas of DBN V.2.6-198:2014 [9] to determine the fictional transverse load and fictional transverse force used in the calculations of attachment of rigid decking, elements of longitudinal or transverse ties to the compressed beam flange. To calculate the fictional loads, the building codes [9] adopt a static model that can be considered flat, since it considers only the forces and strain in the plane of the upper flange. The effects of lateral deformation of the lower flange and spatial twisting of the cross-section are not taken into account in this model.

On the other hand, such a relatively simple representation of the beam operation in the standards allows describing the fictional load as a function of the longitudinal force and strain of the restrained flange, which is obtained in the static model as a result of solving the differential equation of a compressed rod from the condition of balanced forces in the flange. When laterally restrained, the upper flange will move due to the pliability of the attached structures. To take into account the deformation of the upper girder of the beam, its total total value should be used instead of the initial curvature. In this case, it is assumed that the initial curvature of the beam and the displacement of the upper girders are distributed along the length of the beam according to the same law. The maximum fictional transverse load will be determined in this model by the formula:

$$q_{fic} = \frac{\pi^2 N}{l^2}(v_0 + v) \tag{1}$$

where N is the value of the longitudinal compressive force in the beam flange determined in the plastic stage of operation ($N = N_{pl} = A_f R_{yf} + 0{,}25 A_w R_{yw}$, where A_f, A_w are the cross-sectional areas of the compressed flange and beam web; R_{yf}, R_{yw} are the calculated steel resistances of the compressed flange and web, respectively); l is the span of the beam; v_0 is the largest value of the initial curvature of the beam (in Table 5.1 of EN 1993-1-1 it corresponds to the value e_0, which is set depending on the type of curvature curve, cross-section and span of the beam, as well as the type of calculation); v is the largest value of the displacement strain of the compressed flange under load in the y-axis direction.

Next, we will use expression (9.34) of DBN V.2.6-198:2014 [9] for the fictional load:

$$q_{fic} = \frac{3Q_{fic}}{l} \tag{2}$$

The values of the fictional transverse force Q_{fic} in formula (2) can be determined by the approximate dependence (8.14) approximated in DBN V.2.6-198:2014 [9]:

$$Q_{fic} = 7{,}15 \cdot 10^{-6} \left(2330 - \frac{E}{R_y}\right) \frac{N}{\varphi}, \tag{3}$$

where E is the modulus of elasticity of steel; R_y is the calculated resistance of steel to tension, compression and bending beyond the yield strength; φ is the stability factor of under central compression, which is assumed to be equal to one when the compressed beam flange is continuously restrained by the supporting deck.

By equating formulas (1) and (2) with expression (3), we obtain a formula for determining the total allowable horizontal curvature, in which the flexibility of the upper flange fastening is taken into account by assuming an increased initial beam curvature, including strain of the upper flange under load:

$$v_{tot} = 7{,}15 \cdot 10^{-6} \left(2330 - \frac{E}{R_y}\right) \frac{3l}{\pi^2} = \frac{l}{\delta}. \tag{4}$$

Some values of the dimensionless parameter δ for different values of R_y and $E = 2{,}06 \cdot 10^5$ MPa are given in Table 1.

Table 1 — The value of the dimensionless parameter δ for determining the permissible curvature

R_y, МПа	δ	R_y, МПа	δ
220	330	300	280
230	321	310	276
240	313	320	273
250	306	330	270
260	299	340	267
270	294	350	264
280	289	360	262
290	284	370	259

Note. When determining the permissible value of the initial beam curvature in the elastic stage, the obtained values should be reduced by the value N_{el}/N_{pl}.

The total curvature of the loaded beam takes into account the influence of transverse displacements of the compliant restraint caused by the action of external and fictional loads. The increase in deformation for unbraced beams usually does not exceed 20% [5]. Therefore, when approximating the permissible value of the initial curvature of an unloaded beam, the values in the table should be increased by 20%.

The initial geometric imperfections that occur in a damaged beam in service can be determined during inspections of rehabilitated buildings by measuring the actual deviations using tools. In this way, the values of the initial curvature strain at the level of the weight center of the cross-section, as well as the initial angle of rotation, are determined in the first place. The total linear strain of the initial curvature at the level of the beam upper flange can be determined approximately from the second equation shown in Fig. 5. The initial deflection of the beam is practically absent due to its significant stiffness in the main plane.

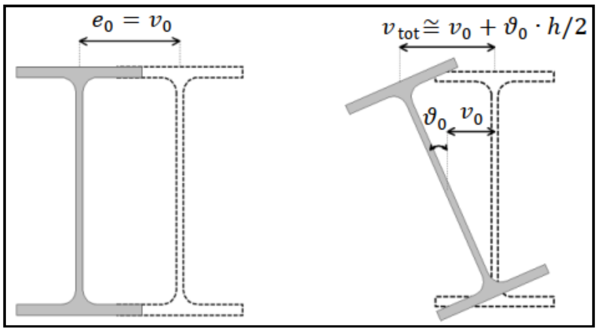

Figure 5 – Determination of geometric imperfections, occurring in the operated damaged beam

Separately, it is necessary to focus on the study of the interaction between damage and stresses in the restrained elements. If the actual curvature exceeds the permissible value established by formula (4) and Table 1, a more detailed analysis of the stress-strain state of the beam should be performed. It is already necessary to determine the additional normal stresses due to the initial curvature of the beam and compare them with the permissible stress increase, i.e., the difference between the value of the calculated resistance R_y, or the actual yield strength σ_y obtained as a result of the inspection tests, and the value of normal stresses in the uncurved beam σ. The methodology for determining additional stresses, which is given in [10], was implemented in the state standard for assessing the technical condition of steel building structures in operation [11]. Its prerequisite is to take the curve corresponding to the curvature function as a square parabola. It is worth noting that the disadvantages of the methodology are the failure to take into account the level of load application along the height of the beam and the existing eccentricities in asymmetric cross-sections, which is not a safety factor. A refinement and comparison of this methodology are given in the scientific article [8]. The form of the formulas in Table 2 is slightly modified to take into account the flange displacement under load and is intended only for a symmetrical I-beam.

Table 2 – Additional normal stresses in a beam with a curvature

Concentrated force in the middle of the span
$$\sigma_{ad} = \frac{2Fv_{tot}}{l} \cdot \frac{E\omega}{GI_t}\left(1 - \frac{\tanh\frac{kl}{2}}{\frac{kl}{2}}\right)$$
Uniformly distributed load
$$\sigma_{ad} = qv_{tot} \cdot \frac{E\omega}{GI_t}\left(1 - \frac{\tanh\frac{kl}{2}}{\frac{kl}{2}}\frac{\tanh\frac{kl}{4}}{\frac{kl}{4}}\right)$$

Note. F is a concentrated force; q is uniformly distributed load; G is the shear modulus; $k = \sqrt{\frac{GI_t}{EI_\omega}}$ is elastic bending-torsional characteristic; I_t is the torsional constant; I_ω is the warping constant; ω is the warping area of the beam cross-section at the stress determination point; $\omega = bh/n$; b is the width of the beam flange; h is the height of the beam; $n = 4$ in the elastic stage of design, $n = 6$ in the plastic stage.

The design model laid down in the European design standards EN 1993-1-1 [12] allows for a variety of loading conditions and types of diagrams, as well as bracing conditions. The initial curvature of the beam is assumed to be the maximum permissible and is determined depending on the type of curvature curve, cross-sectional type, and span of the beam. The calculation method EN 1993-1-1 [12] is more open to the designer and is more intended for machine design. The solution of the corresponding differential equation for the fictional load can also be represented in accordance with the European approach in the form:

$$q_{fic} = \alpha \frac{\pi^2 N}{l^2} v_0 , \tag{5}$$

where α is a multiplying factor to account for the second-order effects, which in the compressed rod model is determined by equation:

$$\alpha = \left(1 - \frac{1}{\alpha_{cr}}\right)^{-1} , \tag{6}$$

where α_{cr} is the coefficient of increase in the design load at which the buckling in the elastic stage occurs.

In equation (5), the longitudinal compressive force in the upper beam flange is defined in the elastic stage as the ratio of the bending moment that causes it to the distance h_f between the weight centers of the beam flanges, taking into account the longitudinal force arising in the beam from the external load:

$$N = N_{el} = \frac{M_{y,Ed}}{h_f} - \frac{N_{Ed}}{2} , \tag{7}$$

where $M_{y,Ed}$, N_{Ed} are the design values of bending moment relative to the *y-y* axis and longitudinal force.

The critical load on the upper flange will be proportional to its bending stiffness according to the Euler's formula. Since, when taking into account the pliability, the critical force in the flange will be approximately equal to the shear stiffness of the structures attached to it [5], a simplified expression can be written to determine the total strain of the initial curvature at the level of the upper flange of the restrained beam:

$$v_{tot} \approx v_0 \left(1 - \frac{N}{S}\right)^{-1} = v_0 + \frac{Nv_0}{S - N} , \tag{8}$$

where S is the available shear stiffness of the bracing structures per beam, which must exceed the required stiffness according to formula (BB.2) of EN 1993-1-1 [12] to accept the translational restraint and ensure the beam stability.

If a profiled deck is rigidly attached to the upper flange of a steel beam in lightly roofed framework using self-tapping screws and riveted together at the longitudinal joints (Fig. 6), it can prevent torsion and be used to counteract the lateral-torsional buckling of the beam. This function can also be performed by cross bracings in purlinless buildings for frame rafters.

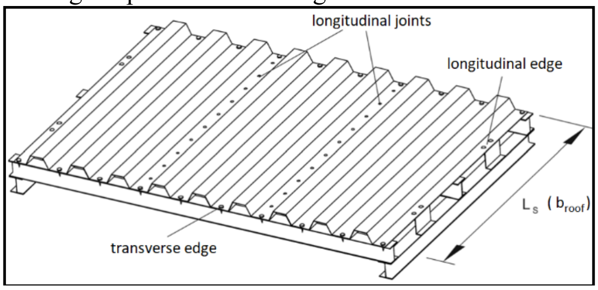

Figure 6 – An example of a roof panel with fastening of profiled deck to the purlins on all four edges

For an approximate determination of the shear stiffness of a profiled deck in newtons, EN 1993-1-3 [13] provides the expression (10.1b):

$$S = 1000\sqrt{t^3}(50 + 10\sqrt[3]{b_{roof}})\frac{s}{h_w} , \tag{9}$$

where t is the thickness of the profiled deck in mm; b_{roof} is the width of the roof in mm (L_s); s is the distance between purlins in mm; h_w is the height of the profiled deck in mm.

The formula (9) slightly underestimates the stiffness value, but it is suitable for any profiled deck and does not require that the deck be fixed to the supporting structures with all four edges, but only with two. This method of fastening simplifies the process and reduces the erection cost. Some manufacturers (Hoesch, Fischer, Arcelor) have developed their own methods for determining the shear stiffness of profiled decking, which can be used only for this product and in the case of fastening with four edges (Fig. 6). For economic reasons, profiled deck is often fixed through a wave (mostly to channel purlins). Formula (9) is used when fixing the deck to the beam in each wave. If the profiled deck is fixed only every second wave, then its shear stiffness is assumed to be reduced by a factor of five compared to the initial stiffness.

Conclusions

The highlighted and considered approaches to determining the permissible curvature of steel I-beams in the horizontal plane can be used with sufficient accuracy in the restoration of damaged building elements. Despite certain drawbacks of these methods, they are quite simple and comply with the design standards for steel building structures.

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