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RESEARCH PAPER

A comparative study of beam design curves against lateral torsional buckling using AISC, EC and SP

Vera V. Galishnikova*, Tesfaldet H. Gebre

Peoples' Friendship University of Russia (RUDN University)
6 Miklukho-Maklaya St., Moscow, 117198, Russia

*galishni@gmail.com

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Abstract. ***Introduction.*** Structural stability is an essential part of design process for steel structures and checking the overall stability is very important for the determination of the optimum steel beams section. Lateral torsional buckling (LTB) normally associated with beams subject to vertical loading, buckling out of the plane of the applied loads and it is a primary consideration in the design of steel structures, consequently it may reduce the load currying capacity.

Methods. There are several national codes to verify the steel beam against LTB. All specifications have different approach for the treatment of LTB and this paper is concentrated on three different methods: America Institute of Steel Construction (AISC), Eurocode (EC) and Russian Code (SP). The attention is focused to the methods of developing LTB curves and their characteristics.

Results. AISC specification identifies three regimes of buckling depending on the unbraced length of the member (L_b). However, EC and SP utilize a reduction factor (χ_{LT}) to treat lateral torsional buckling problem. In general, flexural capacities according to AISC are higher than those of EC and SP for non-compact sections.

Keywords: steel beams, structural stability, lateral torsional buckling, beam design curves

Introduction

Beams are structural elements loaded in a traverse direction, in other way beam may be defined as a member subjected essentially to bending and shear force but its behavior is dominated by its bending deformation [1; 2]. For the design and construction of beam structures different countries have articulated their own codes for laying down the guidelines. This paper is concerned with the method of beam design curves against lateral torsional buckling using AISC (American Institute of Steel Construction), EC (Eurocode) and SP (Russian Code) since lateral torsional buckling is the main limit state that must be checked for steel beams [3–6]. Structural stability is an essential part in the design process for steel structures and checking the loss of overall stability often is very important for determination of the section of steel beams. Lateral torsional instability is normally associated with beams subject to vertical loading buckling out of the plane of the applied loads by deflecting sideways and twisting behavior analogous to the flexural buckling of struts [7–8]. There are various approaches to verify the steel beam against lateral torsional buckling (LTB) and in this paper, the comparison of calculations and methods has been shown according to three different methods: AISC, EC and SP [3; 9]. According to all specifications, yielding and lateral torsional buckling are the two limit states for flexural members. Yielding and lateral torsional buckling is treated separately for clarity of the comparisons. Lateral torsional buckling is a limit state that may assure the strength of a beam [10]. The problem of lateral torsional buckling of steel beams has been studied extensively by many authors, including Trahair and others [11–14]. When a beam is bent about its axis of greatest flexural rigidity, it may twist before it attains its strength limit state. The twisting of the beam goes on once the compression flange becomes unstable due to its being exposed to flexural induced axial stresses and acts like a strut consequently the compression flange will tend to buckle sideways dragging the tension flange with it. Flexural torsional buckling is a primary consideration in the design of steel structures, as it may reduce the load currying capacity. Unless it is prevented either by sufficient bracing or members which have adequate flexural and torsional stiffness's, larger member must be used to avoid premature failure [15]. Once the flange is restrained at intervals, LTB may occur between the restraints and this must be checked. If this restraint is continuous, the beam is fully restrained and LTB will not occur. A beam is considered to be unrestrained when its compression flange is permitted to displace laterally and rotate. When an applied load causes both lateral displacement and twisting of a member LTB has occurred. All specifications have different approach for the treatment of LTB and in this article the attention is focused to the methods of developing LTB curves and their characteristics.

Methods

Design according to the AISC approach

The AISC specification provisions for LTB are considered in three different parts of buckling depending on the unbraced length of the member (L_b) [16]. Two threshold values for unbraced length i.e. L_p and L_r are well-defined in AISC specification. The L_p value provides a separating line between plastic (no lateral buckling) and inelastic buckling behavior. Similarly, the L_r value provides a separating line between inelastic and elastic buckling behavior. According to AISC, plastic moment capacity of a compact member can develop if the unbraced length is less than L_p and using this value in design represent the optimum use of steel [16–19]. The member's capacity reduces linearly between M_p and $0.7M_y$ if the unbraced length is between L_p and L_r. If the unbraced length is greater than L_r, then elastic buckling is expected to occur and the capacity can be found using elastic critical buckling moment (M_{cr}). The C_b factor given in design specifications for non-uniform moment diagrams can be used to estimate the increased brace requirements for other loading cases [20]. The following equations are summarized for the nominal moment capacity of lateral torsional buckling as per the AISC specification.

$$M_n = M_p = Z * F_y ,$$

when $L_b \le L_p$,

$$M_n = C_b * \left[M_p - \left(M_p - 0.7S_xF_y\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p ,$$

when $L_p < L_b \le L_r$,

$$M_n = M_{cr} = S_x \frac{C_b * \pi^2 E}{\left(\frac{L_b}{r_{rs}}\right)^2} \sqrt{1 + 0.078 \frac{J}{S`_x h_0}\left(\frac{L_b}{r_{rs}}\right)^2} ,$$

when $L_b > L_r$,

where: $$L_b = 1.7 r_y \sqrt{\frac{E}{F_y}};$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{J}{S_x h_0}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{0.7F_y}{E}\frac{S_x h_0}{J}\right)^2}} ;$$

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} .$$

M_n is the lateral torsional buckling moment, M_p is the plastic moment, F_y is the yield stress of the steel section, S_x is the section modulus of the compression flange about the x-axis, r_{ts} is the radius of gyration of cross-section, h_0 is the distance from the centroid of the top flange to the centroid of the bottom flange, L_b is the unbraced length and L_r and L_p are the two threshold values for unbraced length for the inelastic range and C_b is the moment gradient factor.

Design according to the EC approach

In Eurocode 3, the capacity of a member with respect to the buckling and instability is taken into account by a reduction factor (χ_{LT}) [21–25]. This factor is strongly dependent on the member slenderness parameter (λ_{LT}) [22]. According to Eurocode 3, the beam should be verified against lateral-torsional buckling resistance as follows: The elastic critical moment (M_{cr}) is used as the basis for the methods given in design codes for determining the slenderness of a section. The elastic critical moment (M_{cr}) is similar to the Euler (flexural) buckling of a strut as it defines a buckling load [26]. Euler bucking explains the axial compression that will cause a strut to fail in elastic flexural buckling compared with the elastic critical moment that defines the moment which will result in failure due to elastic lateral torsional buckling of a beam. According to Clause 6.3.2.1(1) of EN 1993-1-1, the beam should be verified against lateral torsional buckling resistance as follows [27–30]:

$$M_{c,Rd} = M_{pl,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{MO}}$$

$$\chi_{LT} = \frac{1}{\Phi + \sqrt{\Phi^2 - \beta\, \lambda_{LT}^2}} ,$$

with $\chi_{LT} \le 1.0$ and $\chi_{LT} \le \left(\frac{1}{\lambda_{LT}}\right)^2$.

And Φ_{LT} is defined,

$$\Phi_{LT} = 0.5[1 + \alpha_{LT}\,(\lambda_{LT} - \lambda_{LT,O}) + \beta\, \lambda_{LT}{}^2\ \lambda_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}},$$

where α_{LT} is the imperfection factor corresponding to the appropriate buckling curve; γ_{MO} is the partial factor for member instability which has a recommended value of 1.0 in EC3; $W_y f_y$ is the section moment resistance; λ_{LT} is the modified slenderness, and the values of α_{LT}, β and depend on the type of beam section.

Design according to the Russian Code approach

Depending on the purpose and conditions of the structures, calculation of flexural elements (beams) should be performed without taking into account or taking into account plastic deformations in accordance with the subdivision of elements into three classes. Beams of the first class should be used for all kinds of loads and be calculated within elastic deformations; Beams of the second and third classes should be used for static loads and taking into account the development of plastic deformations [3; 31]. This approach also used a reduction factor χ_{LT} to treat lateral torsional buckling problem [22]. Using SP the nominal moment capacity is suddenly drops from plastic moment capacity in non-compact section and it is limited to a small lateral bracing length. For Class 2 and Class 3 members, if the member is loaded with moment in one of the principal plane only, the design buckling resistance moment (nominal moment capacity for LTB) should be calculated as follow:

$$M_{b,RD} = \chi_{LT}\, W_y\, f_y \gamma_c\ ,$$

when $\chi_{LT} \geq 0.85$, the section is in the elasto-plastic stage.

As the result, the Young modulus declines and the buckling factor has to be modified [3; 31]. The modification of buckling factor is specified in SP code and this is done by finding the coefficients α (section SP16 G.4) and ψ (SP16 Tables G.1 and G.2). The buckling factor for members with doubly-symmetric *I*-sections is calculated as follows.

$$\chi_{LT} = \begin{cases} \varphi_1\,, \text{if } \varphi_1 \ \leq 0.85 \\ 0.68 + 0.21\varphi_1, \text{if } \varphi_1 \ > 0.85 \end{cases},$$

where φ_1 is defined in the SP code (section SP 16 G1).

The Russian standard also provides equations for stable length limits of the beam[1]. The over all stability of the flange is ensured if the characteristic nominal slenderness is less than the ultimate slenderness value which can be calculated from Ultimate nominal slenderness [SP 16.13330.2011, Chapter 8.4.4, Table 11] below: for condition of Upper flange we can consider the following formula.

$$\frac{l_{ef}}{b_f} < (0.35 + 0.0032\,\frac{b}{t} + \left(0.76 - 0.02\,\frac{b}{t}\right)\frac{b}{h})\sqrt{\frac{R_y}{E}}\ ,$$

but $$\begin{Bmatrix} 1 \leq \frac{h}{b} \leq 6 \\ 15 \ \leq \ b/t \ \leq \ 35 \end{Bmatrix},$$

where b and t are width and thickness of the compression flange; h is distance (height) between the axes of the flanges.

When b/t < 15, need to take the value b/t = 15. If the limit slenderness which is flange stability is more than the limit value, it is necessary to install the intermediate stiffeners for reduction of the effective length l_{ef}. Under the action of normal and tangential stresses, the beam wall can lose local stability, i.e. its local buckling can occur.

Results and discussion

The curves of nominal flexural strength for 60Ш2 and W12×30 of the steel beam sections, according to the AISC specification, show the distribution of capacity of the steel sections across a wide range of lateral bracing length (L_b), shapes of the moment with C_b = 1 and with a value of F_y = 275 MPa. The graphs are shown in figures 1 and 2.

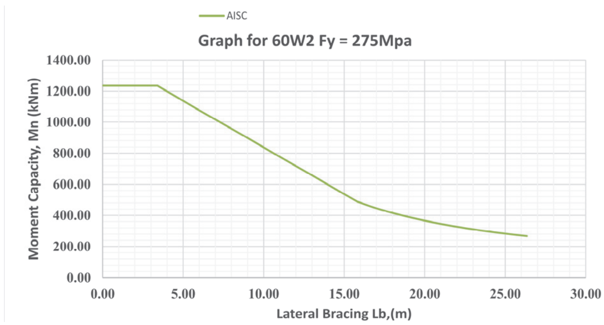

Figure 1. ФM_n vs L_b for 60Ш2 according to AISC

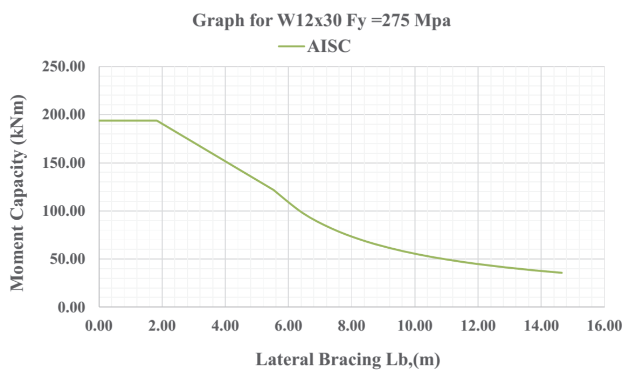

Figure 2. ФM_n vs L_b for W12×30 according to AISC

[1] SP 16.13330-2017. Building Codes. Design of Steel Structures. Moscow, 2017. (In Russ.)

Similarly, curves of the nominal flexural strength of sections for 60Ш2 and W12×30 of the steel beam sections, using Eurocode, show the comparison of capacity of the steel sections across a wide range of lateral bracing distances (L_b) and with a value of F_y=275 MPa. The graphs are shown in figures 1 and 2.

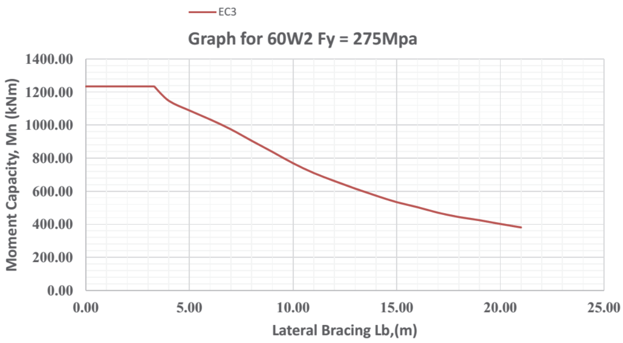

Figure 3. ΦM_n vs L_b for 60Ш2 according to EC

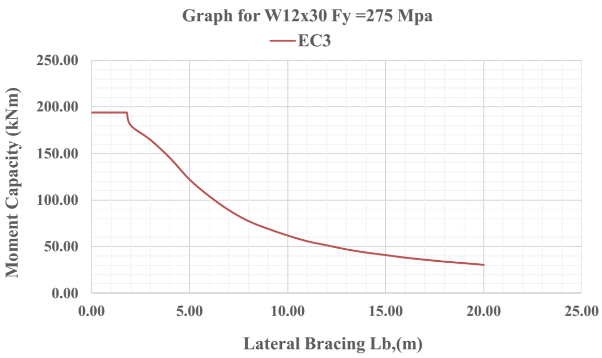

Figure 4. ΦM_n vs L_b for W12×30 according to EC

Likewise, according to Russian Steel Construction Specification, curves of the nominal flexural strength for 60Ш2 and W12×30 of the steel beam illustrate the comparison of capacity of a single steel section across a wide range of lateral bracing distances length (L_b), and with a value of F_y =275 MPa. The graphs are shown in figures 5 and 6.

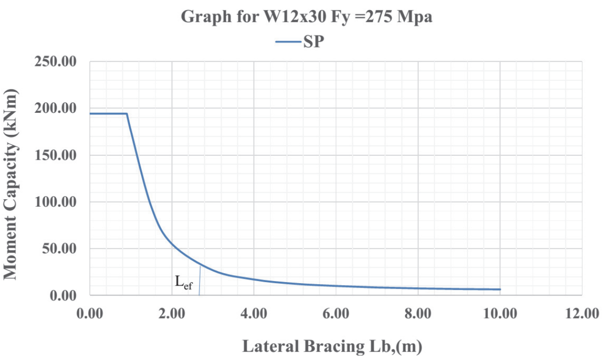

Figure 5. ΦM_n vs L_b for W12×30 according to SP

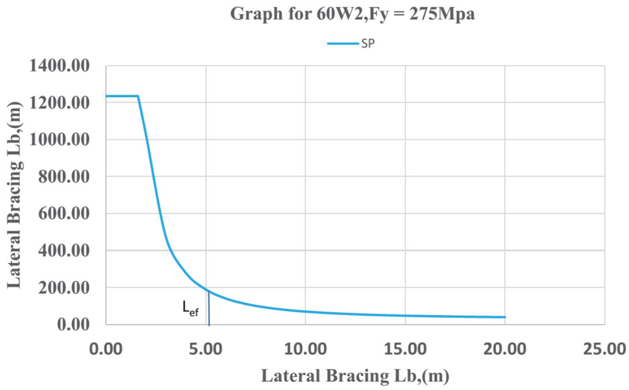

Figure 6. ΦM_n vs L_b for W12×30 according to SP

A combined graph for 60W2 and W12×30 of rolled *I*-shaped beam sections were considered to compare lateral torsional buckling capacity curve of member according to three specifications: America Institute of Steel Construction, Eurocode and Russian Code. A curve of the nominal flexural strength of the steel beam sections together shows the comparison of capacity of a single steel section across a wide range of lateral bracing distances (L_b), shapes of the moment with C_b = 1 and with value of F_y = 275 MPa. The graphs are shown in figures 7 and 8.

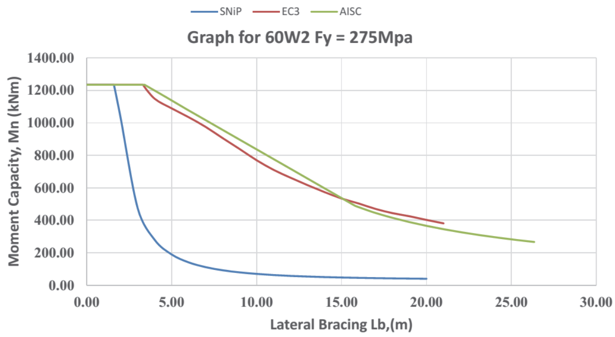

Figure 7. ΦM_n vs L_b for 60Ш2 according to AISC, EC and SP

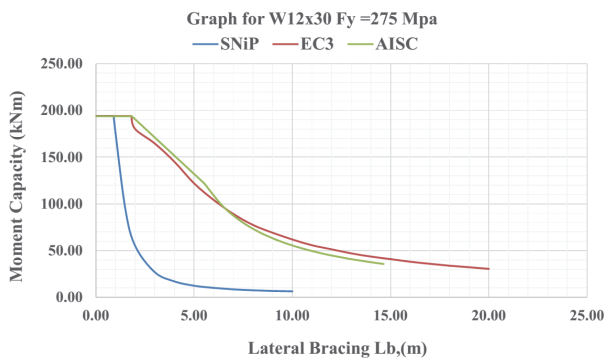

Figure 8. ΦM_n vs L_b for W12×30 according to AISC, EC and SP

As per the result, it has been observed that, there are variations in the values of the strength obtained by three different codes because of the variations in the values of the constants considered by the each code. AISC gives higher capacity in inelastic region but Eurocode 3 gives higher capacities in elastic region. All specification have different approaches for laterally unsupported flexural members, AISC has three regimes of buckling depending on the unbraced length of the member (L_b). However, EC and SP utilize a reduction factor (χ_{LT}) approach to treat lateral torsional buckling problem.

Conclusions

The results of the study showed that, for laterally supported flexural members with compact webs, the fundamental difference between all specifications is the treatment of flange buckling. According to all specifications, the member can reach to its plastic moment capacity if the flanges are compact. Treatment of noncompact flanges is similar to the treatment on noncompact webs in all specifications. According to the AISC specification, the nominal moment capacity reduces linearly with an increase in the flange slenderness and varies between the plastic moment capacity (M_p) and the yield moment considering residual stresses ($0.7M_y$). On the other hand, the nominal moment capacity is equal to the yield moment for Class 3 sections according to the Eurocode 3 and class 1 for SP specifications. In the case of SP, the nominal moment capacity is suddenly drops from plastic moment capacity for non-compact section and as a result, it is limited to a small lateral bracing length. For slender flange members the AISC specification utilizes the elastic critical buckling moment approach. In EC and SP, the post buckling reserve strength approach is utilized and effective cross-section properties are utilized for this purpose. In general, AISC gives higher capacity in inelastic region (non-compact sections). However, Eurocode 3 gives higher capacities in elastic region (slender sections) according to the sample used sections in the paper.

For laterally unsupported flexural members, all specification have different approaches. AISC specification identifies three regimes of buckling depending on the unbraced length of the member (L_b). However, EC and SP used a reduction factor approach to treat lateral torsional buckling problem. In general, flexural capacities according to AISC are higher than those of EC and SP for non-compact sections. Particularly SP approach is lower for flexural capacity and it will be uneconomical approach comparing with the other two approaches.

The design according to a standard makes the analysis process easier and saves time of an engineer. Also, the expertise process becomes clearer. However after the comparison of all standards it can be concluded that design according to the Russian norms is more time consuming and requires competence and great knowledge in the engineering field. In AISC and EC, the analysis process is more precise and it has its own logic and algorithm. It will be easier for a young specialist to use the AISC and EC standards instead of the SP. Besides that, nowadays the Russian standards are more understandable and readable for foreign engineers because of standard harmonization, which is focused on updating the Russian norms and it encourages specialists to keep abreast of new technologies.

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About the authors

Vera V. Galishnikova – Associate Professor, Director of the Department of Civil Engineering, Engineering Academy, Peoples' Friendship University of Russia (RUDN University) (Moscow, Russian Federation). *Research interests:* computational civil engineering, building information modeling, topological computer models of buildings, computational geometry, computational mechanics of complex steel structural systems – latticed plates and shells, thin-walled plate and plate-rod structures, nonlinear finite element analysis of space frames, nonlinear stability of structures. *Contacts:* e-mail – galishni@gmail.com

Tesfaldet Hadgembes Gebre– PhD student of the Department of Civil Engineering, Engineering Academy, Peoples' Friendship University of Russia (RUDN University) (Moscow, Russian Federation). *Research interests:* computational civil engineering, frame stability analysis, lateral torsional buckling, computational mechanics of buckling systems. *Contacts:* e-mail – tesfaldethg@gmail.com

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НАУЧНАЯ СТАТЬЯ

Сравнение расчетных кривых балки с боковым крутильным изгибом с использованием AISC, EC и СП

В.В. Галишникова*, Т.Х. Гебре

Российский университет дружбы народов

Российская Федерация, 117198, Москва, ул. Миклухо-Маклая, 6

*galishni@gmail.com

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Цель исследования. Расчет на устойчивость является неотъемлемой частью проектирования стальных конструкций. Он очень важен для определения оптимального поперечного сечения стальных балок. Поперечное боковое выпучивание обычно происходит у балок, которые подвержены вертикальной нагрузке и теряют устойчивость из плоскости приложения нагрузок. Это является основным фактором при проектировании стальных конструкций и может привести к снижению несущей способности.

Методы. Существуют различные методы расчета стальной балки на поперечное боковое выпучивание. Все нормы расчета по-разному подходят к исследованию поперечное-бокового выпучивания, в данной статье внимание сконцентрировано на трех из них. Первый метод предложен Американским институтом стальных конструкций (AISC), второй описан в Еврокоде (ЕС), третий приводится в российских строительных правилах (СП). Особое внимание уделено методам построения кривых для поперечного бокового выпучивания и определения их характеристик.

Результаты. Нормы, разработанные Американским институтом стальных конструкций, рекомендуют рассматривать три режима потери устойчивости, зависящие от длины элементов (L_b). Однако ЕС и СП дают уменьшение χ_{LT} и предохраняют конструкцию от поперечного бокового выпучивания. В основном изгибная жесткость для поперечных сечений с высокими стенками согласно AISC выше, чем в ЕС и СП.

Ключевые слова: стальные балки, устойчивость, устойчивость плоской формы изгиба балок, форма потери устойчивости

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Об авторах

Галишникова Вера Владимировна **–** доктор технических наук, директор департамента строительства Инженерной академии, Российский университет дружбы народов (Москва, Российская Федерация). *Область научных интересов:* вычислительная строительная инженерия, информационное моделирование зданий, топологические компьютерные модели зданий, вычислительная механика сложных стержневых систем, нелинейные конечно-элементные модели и программные комплексы для расчета пространственных стержневых систем, нелинейная устойчивость конструкций. *Контактная информация:* e-mail – galishni@gmail.com

Тесфалдет Хадгембес Гебре – аспирант департамента строительства Инженерной академии, Российский университет дружбы народов (Москва, Российская Федерация). *Область научных интересов:* гражданское строительство, расчеты рамы на устойчивость, плоская форма изгиба, вычислительная механика. *Контактная информация:* e-mail – tesfaldethg@gmail.com.

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