Tension Field of I-beam under the Action of Bending-shearing

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Abstract
Cutting and damaging the web beam can be divided into three stages, including pre buckling stage, post buckling stage and failure stage. A lot of research work has been shown that the ultimate bearing capacity of the web is still quite limited. At present the theoretical and experimental research on the I-beam web limit bearing is not enough. There is no right to consider the post buckling strength of tapered web anti shear bearing capacity formula. Through the analysis and experimental study about bending shear under the action of I-girder web limit bearing capacity of the buckling stress, we can find that the tension field model of Davies has the highest relative accuracy, the buckling coefficient of various analysis of the calculation precision of the model are significantly improved in this study.
Key words: TENSION FIELD, I-BEAM, BENDING-SHEARING, LIMIT BEARING CAPACITY

1. Introduction

Steel beam is widely used in industrial and civil buildings. In the field of building construction, steel is mainly used for the multi-storey and high-rise housing floor beams, factory work platform beam, crane girder and girder wall and roof system of RIN. In other fields of civil engineering, steel beam also has a wide range of applications, such as a variety of large span bridges, water conservancy structure in the steel gate, etc. According to the method of production, the steel beam can be divided into two types: steel beam and composite beam. Steel beam is then divided into two kinds of hot rolling steel beam and cold formed steel beam. The hot rolled steel beam mainly includes ordinary hot rolled I-beam, channel steel, hot-rolled steel and hot rolled plain. When the load and the span are too large, the type of steel beam is often unable to meet the requirements of bearing capacity or stiffness, due to the limitation of size and specifications. The so-called composite beam is composed of steel plate combination of steel beams. The vast majority of composite beams are welded. In order to avoid confusion in the name and the steel and concrete composite beam, it is also known as the plate beam. The plate girder has two categories: I-section plate girder and box girder. As a result of the production of convenience and rational use, it has been already widely used.

For the web of I-beam, in order to prevent the occurrence of local buckling of stiffener, it is usually arranged in pairs on both sides. The stiffening rib and flange will be separated from the web into a rectangular area of a plurality of supports. Speaking from the stiffness of stiffener and the web, at both sides of the rectangular grid is generally regarded as a simply supported and flange of stiffness on the web turn certain constraints. But we should consider how to consider this constraint. The standards of different countries are not the same. The local stability of the I-beam web is essentially the buckling problem of the local load, shear force and bending moment of the rectangular plate. This paper focuses on the ultimate bearing capacity of web beam shear problem.

Southwell & Skan first got the buckling coefficient of the long and narrow quadrilateral plates under the shear force, that is, k=5.34. Timoshenko used the energy method to study the shear buckling of rectangular plates with four edges simply supported by finite length. And the shear buckling coefficient of the rectangular plates with different width and depth ratio was also obtained. Stein & Neff was modified by the Timoshenko theory, which not only considers the buckling modes of the rectangular plates, but also gives a more accurate result. The shear buckling coefficient of the rectangular plates with arbitrary width and depth ratio could be obtained by the coefficient curves given by them. Budiansky & Connor obtained the shear buckling coefficient of the four side clamped plate by using the method of the inscription. Bulson gave the shear buckling coefficient curve of the two sides simply supported and clamped rectangular plates.

When the external load is greater than the elastic buckling load, the bearing capacity of the upper word beam can still be improved. That is called the strength of a certain post buckling strength. If you neglect the post buckling strength, then you can get very conservative results. At present, most of the relevant provisions of foreign specifications (EC3, AISC, ISO, BS5950) are used for the buckling strength of the web. Mauoi & Skaloud and Johannson introduced the theoretical background and characteristics of the structure design of the plate structure in the Part1.5 of Eurocode3. The design principles of the bending moment, shear force and local load of the web with the stiffening rib and the no stiffening rib were described in detail. This method has become the trend of the development of steel structure design. This design principle has also introduced in the design specification for steel structures of China, which was considered in the design of composite beams subjected to static loads, but the GB50017-2003 was not considered in the post buckling strength.

For local load I-beam limit bearing capacity, in the past three or four years, many scholars were studied on this issue, which mainly including Bergfelt, Roberts, Granath and Lagerqvist et al. Through the experimental study obtained similar results, they thought the destruction form of I-beams under partial load consists of the following web yielding, web buckling or bending. What kind of failure mode was controlled by the geometric size of the beam and the yield strength of the flange and the web? In fact, buckling or folding of the web was not a clear boundary, since the emergence of the two forms of damage. The web would obviously plane deformations. The local load I-beam affecting the ultimate bearing capacity of the biggest factor was the thickness of web and the web of yield strength, the height of the web also had certain influence. The later development of the computational model and the influence of the flange were also taken into account. Taking into account the comple- xity of the problem, including the complexity of the form of damage, the complexity of the various parameters, it was difficult to accurately calculate the ultimate bearing capacity. The current use of the formula was experience and half experience. Previously used methods was to use different formula to calculate the I-beam limit bearing capacity, a web yielding and a web buckling or bending. The latest European specification used the same set of formulas to calculate the ultimate bearing capacity of several possible forms of failure.

2. Methodology

2.1. Definition and Research status of I-beam

Steel structure is an important structural form in architectural engineering. Because of the high strength and high strength of steel, the steel structure members are generally made of thin and thin walled members. With the development of national economy and urban construction, steel structure in the heavy steel structure high-rise, super high-rise buildings and large tower boiler steel frame have been widely used. With the expansion of the use of the building, the structure of the load has been doubled, so as the main bearing and bending member, the beam's cross-section size continues expanded, the height also gradually increased, in order to reduce its span ratio. In the existing building structure member section height can reach 1-2m, plate in the tower boiler steel frame beam section height can reach 8m, these components of the span to depth ratio in 3-10m. At this time, members of steel structure became short, deep components and shear deformation effects became significant. In recent years, the mechanical properties and engineering application of the short deep beams have been studied and tested in many years. Some valuable results are obtained.

Foreign Studies on deep beams are mainly concentrated in the aspects of reinforced concrete deep beams and steel plate shear wall. In 1973, Japanese scholars Takahashi first carried out the experimental study on the steel plate shear wall, and combined with the finite element method to analyze and verify the results of the test. Then the research workers began to carry out in-depth research on the steel plate shear wall. In 1979, Kahn and Hanson proposed the frame structure of the reinforced concrete deep beam to be used to study the anti lateral force performance of the reinforced concrete frame structure. In 1999, Kabele established a reinforced concrete frame structure model of the concrete filled fiber reinforced concrete in the research of structural reinforcement. Kanda improved the efficiency of the production and installation of the structure and the replacement of damaged components by using the precast concrete deep beam and the reinforced concrete frame beam. In 2003, Kesner used fiber reinforced concrete deep beams in the research of seismic strengthening of steel frame, and multiple parameters of single fiber reinforced concrete deep beams were analyzed. In the same year, Toko Hitaka and Chiaki Mastsui proposed the concept of the deep beam with a slit steel shear wall, and then a lot of scholars joined to discuss this problem.

2.2. Research Background on Tension Field Theory of I-beam

Since the 20's of last century, more and more scholars began to study the shear ultimate bearing capacity of the word beams, including Basler, Porter & Rockey, Calladine, Hoeglund, Lee, et al. They used different assumptions and proposed a different approach, which was used by many countries.

Basler and Thurlimann are the earliest scholars who proposed I-beam ultimate shear bearing capacity calculation method of the scholars [1]. In the case of the shear force, the main tensile stress and the principal compressive stress in the 45 degree and 135 degree directions were generated by the same magnitude of the web. After buckling of the web, it was assumed that the principal compressive stress was not increased, but the main tensile stress could still be increased to form a new bearing mechanism. They assumed that the flange of the I-beam weak could not bear tension field transfer lateral loads, that ignored the role of perimeter frame, that tension field only anchor in the transverse stiffeners, which is shown in Figure 1.

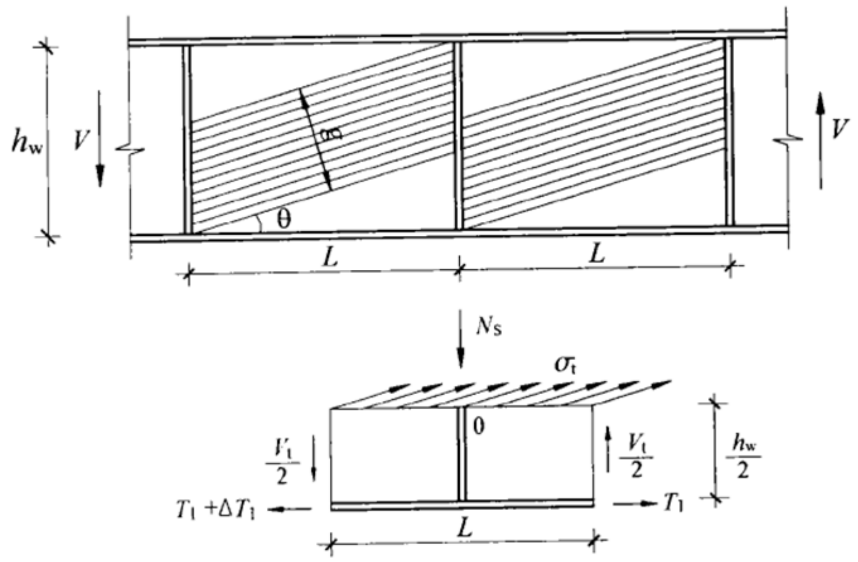


Figure 1. Computational model of Basler's theory

Wilson firstly discussed research on the post buckling behavior of web beam. Wagner explained the tension field effect by using the method of analysis. The theoretical and experimental study of Basler and Thllrlimann were carried out, and the practical method to consider the performance of the web panel with the joint action of bending and shear was proposed. The method was adopted by the AISC1997 standard of USA. Rockey was the representative of the institutional framework mode tension field theory that flange stiffness should be considered [4]. With the belt tension yield, the beam could bear larger load, until the web tension was imposed on the flange lateral film tensile stress in the flange and pull collapse in the flange away from supporting transverse stiffeners at one point due to the beam bending plastic hinge, flange collapse inwards. It was really in the limit state [4]. In this case, the truss beam model must be superimposed on the frame effect of the flange and the lateral stiffening ribs. Rockey tension field theory has been adopted by the British bridge specification BS5400 [2].

Rockey and Skaloud firstly considered the influence the beam flange shear on ultimate bearing capacity. It was considered that the I-beam upon reaching the ultimate bearing capacity of state would form the mechanism shown in Figure 2. Tension and compression flange were respectively formed on the two plastic hinges. They made a series of three tests, each family only changed flange size, and assumed that web buckling load of would not change since the change of the flange. The joist bearing abi- lity of the change was due to changes in the tension band range and angle. It is found that the change of the flange has a great influence on the ultimate bearing capacity. In addition, the position of plastic hinge on the flange of the tension and compression flange is changed with the change of bending rigidity. When the bending stiffness was very large, the inner plastic hinge appeared in the middle of the flange, that is, c=0.5L.

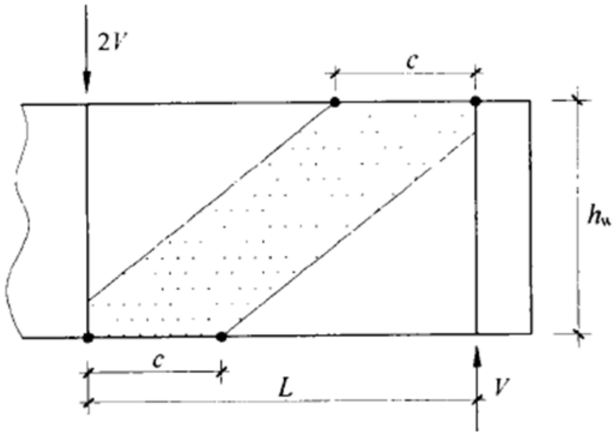


Figure 2. Rockey model

The classical I-beam shear capacity calculation method was represented by the tension field theory of Basler and Rockey [1, 4]. The former formula was simple, but the contribution of the flange to the web was not considered. The classical tension field theory was not considered when calculating the elastic buckling of the web. Later, Calladine proposed a plastic design method [3]. For the elastic buckling load of the web, it was assumed that the web could be replaced with a series of parallel, so that the web would be destroyed. The plastic hinge was formed on the flange. The position of plastic hinge and the inclination of the film tensile stress were related to the rigidity of the wing. However, the method of Calladine could not be used to calculate the beam with a certain bearing capacity before the buckling of the web.

The calculation value of the tension field theory was large, and the ratio of the test value was small, the fact that the structural ultimate bearing capacity was reduced by the initial imperfection. The relationship between the average shear stress of the structure under the limit state was obtained by using the energy method and the static balance method. The calculation accuracy was limited. In the web, the I-shaped beam shear buckling flange and the horizontal stiffening rib formed framing effect was very obvious, the framing effect made the web anti shear bearing capacity [5]. Davies collected previous scholars about I-beam ultimate shear bearing force test data, and the existing calculation method and the test results were compared considering the flange rigidity of influence of I-beam ultimate shear bearing capacity [6].

From the above analysis, it is known that Eurocode 3 simple buckling method does not take into account the flange of I-beam anti shear limit bearing capacity of the effect [7]. Simple buckling from Davies proposed correction method despite the flange self bending stiffness on the ultimate bearing capacity of the contribution.

3. Results and Discussion

3.1. Beam Buckling under Shearing

The basic function of the web is to maintain the spacing between the upper and lower flanges in order to provide a greater moment of inertia, but the web will therefore bear the shear force of the moment balance. In order to prevent premature buckling, the web is divided into several zones in order to improve the shear buckling load of web [8]. The critical buckling stress of rectangular plates under shear force is usually defined by the following formula:

$$\tau_{cr} = k_{cr} \frac{\pi^2 E}{12\left(1-\mu^2\right)} \left(\frac{t_w}{h_w}\right)^2 \tag{1}$$

In this k_{cr} is the shear buckling coefficient, E is the modulus of elasticity, μ is Poisson's ratio. h_w and t_w are the height and thickness of the plate.

In the provisions of AISC and AASHTO, the ultimate shear strength of the web buckling load. It is assumed that the transverse stiffening rib has enough rigidity to ensure the buckling of the sinusoidal wave in the web, and to provide a simple support for the web. Therefore, the shear buckling coefficient of the simply supported plate is used to calculate the elastic buckling load of the web. The tension field theory from Porter & Rockey is also adopted for such a hypothesis. But this assumption is reasonable. Table 1 shows the comparison of the shear buckling coefficients of the rectangular plates with different aspect ratio. And the data comes from Galambos' study [10].

Table 1. Shear buckling coefficient of rectangular plate

Boundary conditions	Web height ratio							
	0.50	0.70	0.90	1.00	1.50	2.00	3.00	∞
Simply supported rectangular plate	25.36	14.90	10.59	9.34	7.12	6.34	5.78	5.34

Clamped rectangular plate	26.74	16.63	13.27	12.60	10.88	10.13	9.53	8.98
Percent (%)	5.44	11.61	25.31	34.90	52.81	59.78	64.88	68.16

From the above table it shows that with the increase of L/h_w, the difference is rapidly expanding. The flange of the I-beam provides actually on the web a flexible constraint. It is assumed that the plate with four edges simply supported, when the big ratio of width to height will get very conservative results. Chern & Ostapenko considered that the ultimate bearing capacity of the beam was assumed to be fully supported by the flange [9]. In the steel structure design specification GB50017-2003, the embedded coefficient x is introduced, which considers the constraint of the flange to the web, but it is the same for all the steel beams.

3.2. I-Beam buckling under the Action of Bending and Shearing

At the same time, it is necessary to consider the influence of bending and shearing on the stability of the web of the beam at the same time. Chen Shaofan gives the critical conditions for the elastic buckling of a quadrilateral support plate subjected to both bending moment and shear force:

$$\left(\frac{\tau}{\tau_{scr}}\right)^2+\left(\frac{\sigma_b}{\sigma_{bcr}}\right)^2=1 \qquad (2)$$

In this formula, τ and σ_b are the critical values of the shear stress and the maximum bending normal stress. σ_{bcr} and τ_{scr} are respectively the critical values of the shear stress and bending stress.

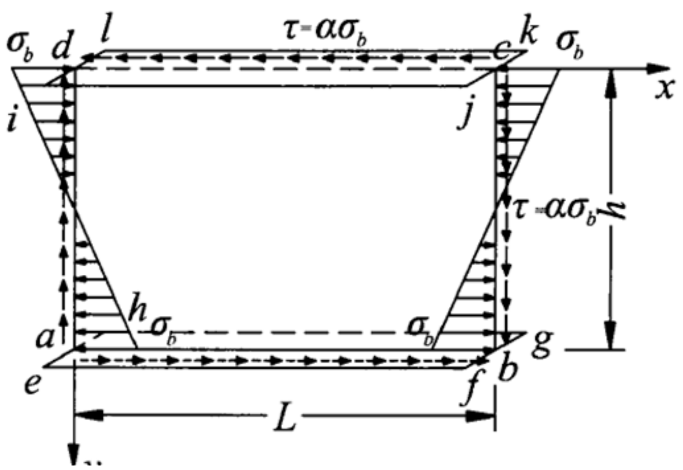


Figure 3. A schematic diagram of the web under the action of bending and shearing

The elastic buckling analysis is carried out in the case of the shear stress and bending stress at the same time which is shown in Figure 3. ANSYS analysis is performed on the web, that is $\tau=\alpha\sigma_b$. Web height $h=800mm$, the width of the flange: $b_f=200mm$, flange thickness t_f: 6~16mm, aspect ratio L/h: 1.0~4.5, Stress ratio coefficient σ: 0~1.0. It can be seen that considering the constraint of the flange the buckling critical point of the web is located near the critical curve of buckling plate.

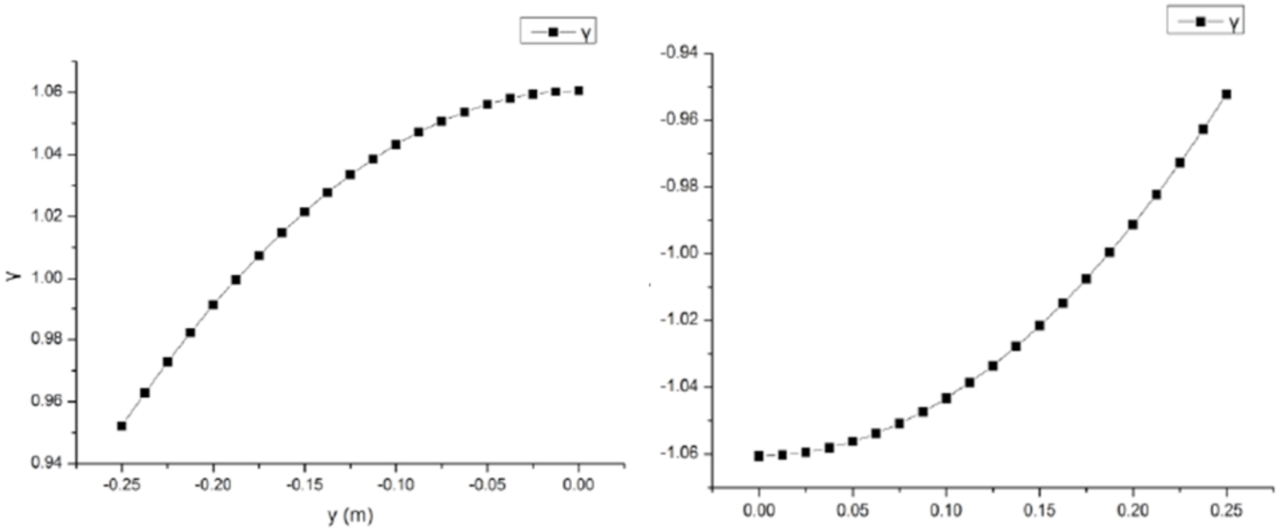
3.3. Analysis of additional stress caused by bending and shearing

In order to analyze the factors that affect the magnitude of the additional stress caused by shear deformation, the shear deformation influence coefficient is obtained:

$$\gamma=\pm\frac{1+v}{\beta^2}\left(2-\frac{4y^2}{3h^2}+\frac{8\alpha}{1+6\alpha}\right) \qquad (3)$$

In order to show the influence of the different variables on the shear deformation γ, the relation curves of the relationship between α and β are drawn. The Poisson's ratio *v=0.3.*

Select $\alpha=2$, $\beta=3$, discuss the variation curves of the coordinates are shown in Figure 4.



(a) (b)

Figure 4. The relation between γ and y

From figure 4, when the shear deformation coefficient y is positive, the shear deformation influence coefficient γ is negative, and the effect of shear deformation is increased with the increase of the shear deformation y, which makes the stress region stress decrease, and more close to the neutral axis shear deformation. Then select $\alpha = 2$, $y = 0$, The relation between γ and β is shown in figure 5.

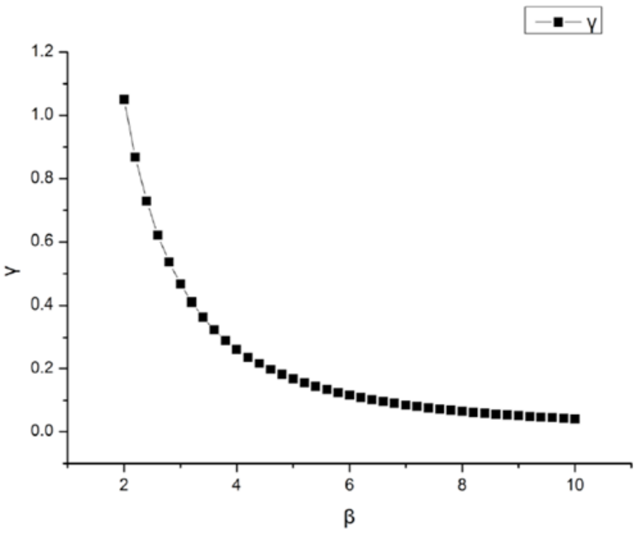


Figure 5. The relation between γ and β

The results show that the influence of shear deformation on the normal stress of the cross section of 4-23 is obvious. It then shows that the influence of shear deformation on the stress analysis of short deep beams cannot be neglected. With the increase of the component ratio, the influence coefficient of shear deformation is gradually reduced. When the cross ratio is greater than 0, the influence coefficient is close to 5. The normal stress of the beam section is consistent with the classical beam theory, which is consistent with the conclusion of the beam theory.

4. Conclusions

The research on I-beam shear ultimate bearing capacity research are summarized in this paper, including the tension of the Basler field method, improved tension field method from Porter & Rockey, Hoglund's stress field theory. Through the comparison with the previous experimental results, the existing methods are discrete. The reason is that these methods are not regarded to be considered as the flange to the web, or the contribution of the flange to the ultimate bearing capacity. Description of I-beam steel flange on the web provides the restrained torsion reasonable parameters in this paper. In Web limit bearing force calculation formula of introducing new buckling coefficient, Davies' simple buckling correction method puts forward the new I-beam ultimate shear bearing force calculation formula. Through ANSYS nonlinear finite element analysis and comparison with existing experimental data, it is proved that the proposed method is less discrete, and is suitable for wide range, especially for general purpose high thickness or wide and high.

Due to more efficient use of material properties and good economic effect, I-beam are becoming more and more widely applied in industrial and civil buildings. I-shaped beams are typically used to design the height of section changes according to the bending moment variation. The section size of the economy and I-beam is often used high and thin webs. However, when the thickness of the web is large, it is easy to lose the local instability, which affects the bearing capacity of the whole structure. In this paper, a general effect of the bending-shearing action on tension field of I-beam is discussed. It makes a brief summary of the research work of the whole paper, and puts forward the idea of further research work in the future.

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References

1. Basler K. (1961) Strength of plate girders in shear. *Journal of Structural Division*, 87(7), p.p. 151-180
2. Fujii T. (1968) On an improved theory for Dr. Basler's theory. Final Report of the Eighth Congress of the International Association for Bridge and structural. Engineering, p.p. 477-487
3. Chern C, Ostapenko A. (1969) Ultimate strength of plate girders under shear. *Fritz Engineering Laboratory Report*, No.328.7, Leigh University.
4. Rockey K C, Skaloud M. (1972) The ultimate load behavior of plate girders loaded in shear. *Structural Engineer*, 50(1), p.p. 29-48
5. Porter D M, Rockey K C, Evans H R. (1975) The collapse behavior of plate girders loaded in shear. *Structural Engineer*, 53(8), p.p. 313-326
6. Davies A W, Griffith D S C. (1999) Shear strength of steel plate girders. *Proc. Instn Civ. Engrs Structs & Bldgs*, 134(5), p.p. 147-157
7. Eurocode 3. (1993) *Design of steel structures. Part 1.1, General rules and rules for building*. The European Union Per Regulation 305/2011.
8. Hoeglund T. (1971) Simply supported thin plate I-girders without web stiffeners subjected to distributed transverse load. *IABSE, Colloquium, Design of Plate and Box Girders for Ultimate Strength*, p.p. 85-97
9. Lee SC, Yoo CH. (1999) Experimental study on ultimate shear strength of web panels. *Journal of Structural Engineering,* 125(8), p.p. 838-846
10. Galambos T V. (1998) *Guide to Stability Design Criteria for Metal Structures. 5th Edn*. John Wiley & Sons.