The Evaluation Of Basler's Theory In Plate Web Post-Buckling

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ABSTRACT

Basler in 1963 provided the well-known theory to explain the post-buckling behavior of the plate webs in girders by using the diagonal tension field. Basler's theory has been adopted in the AASHTO code since 1971, regardless of the drawbacks which were indicated in the past several decades. By means of the finite element analysis combined with the existing experimental shear tests, the present study reveals that for square web panel with the ratio t_f/t_w=4, Basler's predictions correlate excellently with the results from the present FEM models which is perfectly contrary to Basler's model in the physical meaning. This behavior also indicates that the good correlation between the results calculated by Basler's formula with test results is not trustworthy. In case it accepts the existing paradox in Basler's theory, Basler's tension theory can still provide an acceptable prediction in a certain range of parameters, but this tension field is invalid for a long web panel. Finally, the present study will shed light on the controversy about Basler's theory. The results of this study with high web slenderness can be further researched for aerodrome structures.

KEYWORDS: Basler's theory, Post-buckling, Web, Girder, web slenderness

NOMENCLATURE

X, Y, Z = Coordinate system
k = shear buckling coefficient
V_{cr} = shear buckling strength
V_p = plastic buckling strength
V_u = ultimate buckling strength
V_{BP} = post-buckling strength
a = transverse stiffener spacing
b_f = flange width
D = girder depth
t_f , t_w = flange, web thickness
E = Young's modulus
μ = Poisson's ratio

1. INTRODUCTION

Over the years, the web plate in a plate girder is well-known to maintain the relative distance between the top and bottom flanges and to resist the induced shearing force. Therefore, it is common to choose deep girders to obtain a high strength-to-weight ratio. This entails a deep web whose weight is minimized by reducing its thickness.

Consequently, the web panel buckles at a relatively low value of the applied shear loading. The webs are often reinforced with transverse intermediate stiffeners to increase the buckling strength and the web design involves finding a combination of an optimum plate thickness and stiffener spacing that renders economy in terms of the material and fabrication cost.

Although the diagonal tension theory was developed in 1931 by Wagner to assess the post-buckling shear strength of thin panels used in aircraft structures, the post-buckling strength was accounted for directly in the design of plate girders in civil engineering structures until the 1960s. Instead, the post-buckling strength was indirectly reflected in the design of plate girder web panels by lowering the safety factor. In the late 1950s, extensive studies were conducted by Basler and Thurlimann (1959) on the post-buckling behavior of plate girder web panels under shear. Then, these results and other studies (Basler 1961a, b, 1963) AISC (Specification 1963) first adopted the post-buckling strength into its specifications, and AASHO (Standard 1973).

After that, there have been numerous alternative models relative to Basler's were proposed, but Basler's theory still has been adopted widely in civil engineering structures by its ease of use and its good agreement with some of the test specimens.

Basler's Model

Based on Basler's tension theory, the ultimate shear strength was obtained by adding the post-buckling shear strength, $V_{PB(Basler)}$ to V_{cr}, and it can be expressed as:

$$V_{u(Basler)} = V_{cr} + V_{PB(Basler)} = \left[C + \frac{0.87.(1-C)}{\sqrt{1+(a/D)^2}}\right]V_P \quad (1)$$

Where,

$$\text{if: } \frac{D}{t_w} < 1.12\sqrt{\frac{Ek_{Vincent}}{F_w}} \Rightarrow C = 1.0$$

$$\text{if: } 1.12\sqrt{\frac{Ek_{Vincent}}{F_w}} < \frac{D}{t_w} < 1.4\sqrt{\frac{Ek_{Vincent}}{F_w}} \Rightarrow C = \frac{1.12}{\frac{D}{t_w}}\sqrt{\frac{Ek_{Vincent}}{F_w}} \quad (2)$$

$$\text{if: } 1.4\sqrt{\frac{Ek_{Vincent}}{F_w}} < \frac{D}{t_w} \Rightarrow C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek_{Vincent}}{F_w}\right)$$

$$k_{Vincent} = 5 + \frac{5}{\left(\frac{a}{D}\right)^2} \quad (3)$$

Herein, the flanges were assumed to provide zero anchorage to the theoretical tension field in the initial stages of Basler's derivation, therefore the tension field extends only over a portion of the web as in Fig. 1(b). Actually, Eq. (1) was established based on the slope of the tension field ϕ_{Basler} as expressed in Eq. (4). However, ϕ_{Basler} involves Basler's first assumption that a field of uniform tension stresses flows through a web cross as presented in Fig. 1(a)

$$\phi_{Basler} = \frac{1}{2}\tan^{-1}\left(\frac{D}{a}\right) \quad (4)$$

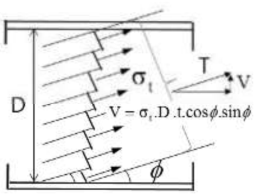

Figure 1.(a) Uniform tension stresses in Basler's theory

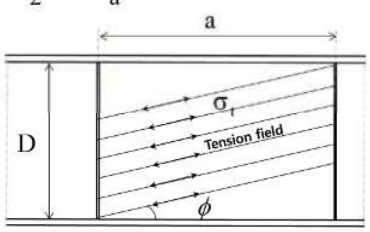

Figure 1.(b) Tension filed over a portion of the web at a failure point in Basler's theory

The above inconsistency is undeniable. However, Basler argued that his theory still provides an acceptable characterization of I-girder shear strengths because of the good correlation between the predicted values by Basler's Eq. and his experimental results.

In addition, the assumption that the flanges are not anchorage to the theoretical tension field leads to the diagonal tension field does not develop near the web-flange juncture and the web collapses after the development of the yield zone as shown in Fig. 2. Apparently, it indicates that the transverse stiffeners must sustain the axial forces. Nonetheless, Lee et al. (2002) revealed that the transverse stiffener is not subjected to the direct compression assumed by Basler (1963), but acts rather like a beam in the post-buckling stage. Moreover, Mai et al proved that the intermediate transverse stiffener acts as a beam since the out-of-displacement of an intermediate transverse stiffener decreases proportionally to the increase in its moment inertia.

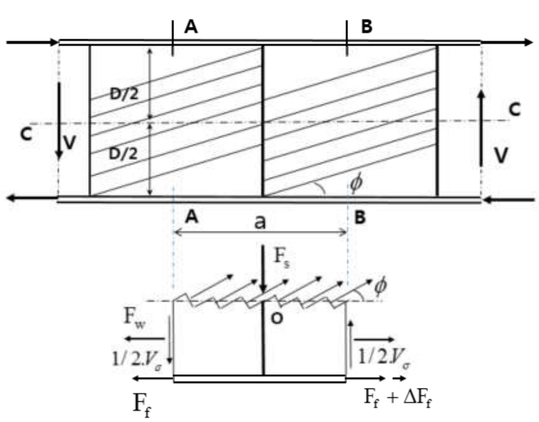

Figure 2. Transverse stiffener resisting diagonal tensions in Basler's theory

Lee and Yoo's Model

In Lee and Yoo's research (1998), based on the comparison of their FEM results with Basler's predictions, Lee and Yoo concluded that Basler's theory gives higher values for the ultimate strength for panels with a/D not greater than 1.5. Lee and Yoo explained that Basler's theory excludes the effect of bending stresses existing in web panels in the post-bucking stage, it gives much higher post-buckling strength, especially for panels with $a/D \leq 1.5$. However, Lee and Yoo observed that Basler's model underestimates the ultimate shear strength for long web panels ($a/D = 3$).

Herein, it is assumed that the effect of bending stresses is truly reason of a difference between Lee and Yoo's results and Basler's prediction, Basler's model must give the same tendency such as that it overestimates ultimate shear strength for all web panels. However, as in Lee and Yoo's above discussion, Basler's model gives two different tendencies for web panels with $a/D \leq 1.5$ and for long web panels. It is seen that Lee and Yoo's above explanation appears questionable.

Alinia et al. 's studies

Alinia et al (2009) studied the shear failure mechanism characteristics on the full-scale plate girder model that include several transversely stiffened plates with an aspect ratio of 1. Comparative analyses on different theories as well as specifications, this research revealed that Basler's results overestimate ultimate shear strength for the girder with the investigated aspect ratio of 1.

In research (2011), Alinia et al realized that the states of stresses on the two faces of the web plate differ, this is due to the secondary bending stresses induced by large out-of-plane deformation on the post-buckling stage. Similar to their results in 2009, Alinia et al concluded that Basler's formula overestimates the ultimate strength of plate girders for aspect ratio =1. Based on Alinia's results, the effect of the secondary bending stresses is obviously a reason for Basler's overestimation of square web panels.

White et al 's study

In an article published in 2008, White et al evaluated the accuracy and ease of use of the existing 12 models for the shear resistance of transversely stiffened steel I-girders. This study used an updated data set from 129 experimental shear tests. The selected tests cover the ranges: $0.5 \leq a/D \leq 3$; $70 \leq D/t_w \leq 800$ with the ratio flange to web falls in the range of $2.5 \leq t_f/t_w \leq 16$.

In this study, White et al confirmed that Basler's model is one of the models excluding an explicit contribution from the flanges to the shear strength. It is true that in the establishment process of the formula, Basler perfectly neglected the contribution of flanges to shear strength. Even if Basler supposed that in practical parameters of girders, the flanges are too flexible to become anchorage to the theoretical tension field in his model. Besides, Basler used a ratio t_f/t_w of 4 for all his test specimens.

White et al' study evaluated the girders which possess flanges in the very wide range of ratio (t_f/t_w), even if it includes the extremely heavy flange case. As a result, White et al concluded that the form of Basler's models implemented in the 2004 AASHTO LRFD Bridge Design Specification and the 2005 AISC Specification give the best combination of accuracy and simplicity for the prediction of the shear resistance.

Although the girder with the light and moderate flanges ($t_f / t_w \le 4$) are objective parameters in Basler's theory, Basler's model overestimates the ultimate shear strength of these parameters as illustrated in Table 1. Nonetheless, a comparison from Table 1 says that Basler's predictions are excellently in agreement with the test results of square specimens with heavy and very heavy flanges. The debate herein is that at a glance Basler's theory seems to predict accurately for a girder with heavy flanges, but a girder with heavy flanges does not suit to Basler's assumption that the flanges of the girder were too flexible to serve as anchorage. In other words, the range of girder parameters that Basler aimed does not correlate well with Basler's prediction, but the girder with the heavy flanges, which seem to correlate excellently with Basler's predictions, are beyond the scope of Basler's investigated girders.

Table 1. Comparison of test results with Basler's prediction for specimens with aspect ratio = 1 and the heavy flanges. Tested parameters refer to Davies and Griffith (1999)

Girder	a/D	D/t	t_f/t_w	V_{ex}/V_{Basler}
TG3	1	400	**6.56**	1.00
TG3.1	1	400	**6.56**	1.00
TG4	1	400	**8.08**	1.15
TG4.1	1	400	**8.04**	1.09
TG15	1	314.4	**5.15**	1.15
TG16	1	314.4	**6.65**	1.26
RTG4	1	267.4	**4.95**	0.97
SD1	1	297.0	**6.00**	1.02
SD3	1	297.0	**6.00**	1.23
TGV1-2	1	289.9	**4.83**	1.06
TGV2-2	1	288.5	**4.83**	1.10
TGV3-2	1	298.5	**4.98**	1.12
TGV4	1	303.6	**5.13**	0.98
TGV5	0.99	302.0	**5.05**	0.97
TGV6	0.99	303.6	**5.13**	0.97
TGV7-2	0.99	302.5	**5.10**	1.03
TGV10-1	0.99	302.5	**5.05**	0.99
TGV10-2	0.99	313.6	**5.24**	1.08
PD1	0.94	800	**10.00**	1.01
PD2	0.94	800	**10.00**	1.01
PD3	0.94	800	**10.00**	1.16
PC3	0.94	800	**10.00**	1.22
			Mean	**1.07**

2. STUDY MODELS

The analyzed model is presented in Fig. 3. The dimensions and material properties were: D = 2032 mm (80 in), $b_f = 610mm, t_f / t_w = 4, \mu = 3$; and E = 210 GPa. The aspect ratio, a/D, and web slenderness ratio D / t_w were the main variables. Herein, the ratio of the flange to the web of 4 is similar to the ratio used in Basler's test. Besides, the models had four $250 \times 3.t_f$ bearing stiffeners placed in pairs at 300 mm from their ends and at load points as well. Intermediate transverse stiffeners were chosen to satisfy that the post-buckling models are single modes that occur within the separate web panels.

Fig. 3 describes the boundary condition and the load of the girders. It was braced in the Y-axis at each of the bearing stiffener locations by simply restricted in the Y direction in the analysis model. The intermediate transverse stiffeners were located at the center of each panel between the bearing stiffeners.

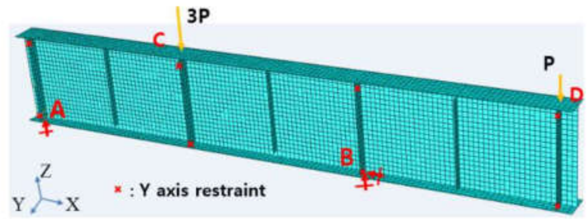

Figure 3. Boundary condition and applied load

Location A was modeled as an ideal roller support where only the vertical displacement was restrained along a line across the width of the bottom flange. The stiffener-flange juncture at location B was restrained by the translation in the X and Z direction. For the applied force, the concentrated loads with a total magnitude of 3P and P were applied at the interior brace location C and at the end brace location D, respectively. Each of these loads was subdivided into three concentrated loads, with three-quarters of the total load placed at the web–flange juncture and one-eighth applied on each side of this position.

2.1. DISCUSSION OF RESULTS

The FEM model in this study accounts for the full rigidity contribution of transverse stiffeners and flanges with the ideal yielding behavior of the material. Thus, it could provide the highest prediction for the ultimate shear strength of web panels. As illustrated for web panels with an aspect ratio of 1 in Fig. 4, there are some of the tests have good correlations with the FEM results and other tests provide less ultimate shear strength than those obtained from the FEM analyses.

Similarly, for long web panels in Fig. 5, we can also see that some tests give results that are in agreement with those obtained from the FEM model. Others provide fewer values than those from the FEM model.

As illustrated in Fig. 6, the present FEM results are compared to those obtained from Basler's formula as well as the existing test results. Surprisingly, for the square web, the results from FEM analyses excellently agree with Basler's predictions. However, the FEM model is totally different from Basler's model in the physical meaning. It can be clearly seen that the present nonlinear analyses were based on the 3D model which includes the significant effect of bending stress in the post-buckling stage. Simultaneously, these results consist of the complete contribution of transverse stiffeners and flanges. Whilst, Basler's theory neglects the bending stress and excludes the explicit contribution of transverse stiffeners and flanges. Definitely, the excellent correlation between Basler's prediction for square web panels and the current FEM results is random. And this argument also says that Basler's good correlation with some of the test results may be contingent behavior.

Table 2. Ultimate shear strength from the FEM results (kN)

Aspect	Slenderness ratio				
ratio	90	120	200	250	300
a/D=1	9100.4	5940.6	2938.7	2193.8	1747.5
a/D=3	8368.1	4954.8	2091.7	1435.4	1041.0

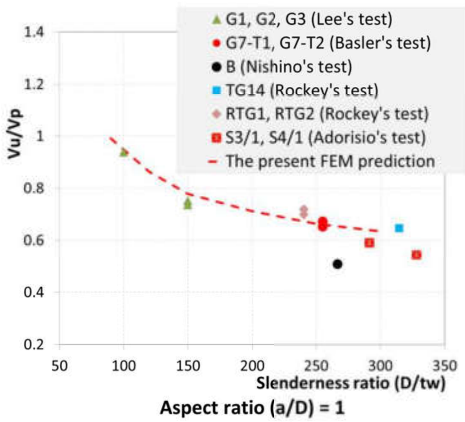

Figure 4. Compared the prediction by present FEM results with test results for aspect ratio =1. (Test results refer to Davies and Griffith (1999))

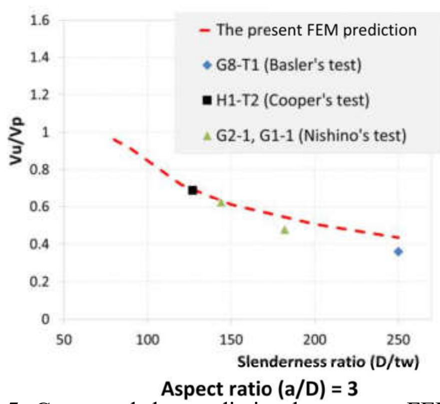

Figure 5. Compared the prediction by present FEM results with test results for aspect ratio =3 (Test results refer to Davies and Griffith (1999))

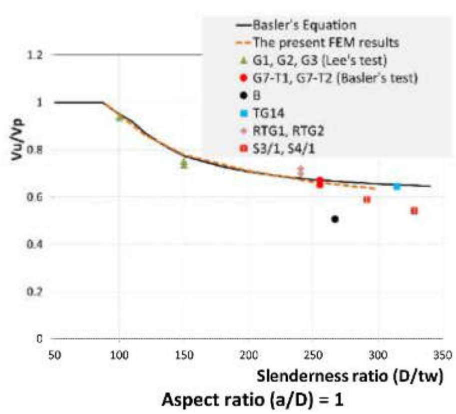

Figure 6. Compared the prediction by present FEM results with Basler's theory for aspect ratio =1. (Test results refer to Davies and Griffith (1999))

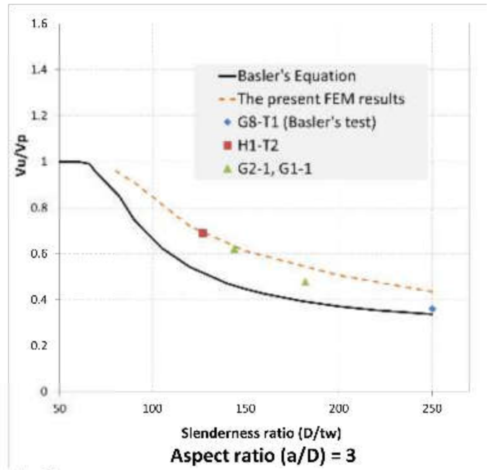

Figure 7. Compared the prediction by present FEM results with Basler's theory for aspect ratio =3 (Test results refer to Davies and Griffith (1999)

By contrast, Basler's theory underestimates all ultimate shear strength for long web panels as shown in Fig. 7. Obviously, Basler's theory provides the different tendencies for square web panels and long web panels. It means that Basler's theory truly appears questionable and the difference in results between Basler's prediction and others, which the prior researchers have observed so far, does not stem from neglecting bending stresses in Basler's theory.

2.2. Developing of post-buckling for aspect ratio =1

For square web panels as an illustration in Fig. 12, it may exist a tension field (membrane stresses) which seen to be similar to the tension field in classical tension theories. However, this tension field could not behave as an expectation in the classical assumption that needs to anchor by the external force. This was pointed out by Lee and Yoo in 2002 and 2006, the anchored mechanism to explain for web post-buckling behavior may be misleading. In other words, the field stresses (membrane stresses) still is observed in the post-buckling stage in present FEM results, and in case it accepts the existing inconsistencies in Basler's theory, Basler's idealization could be accepted for a certain prediction.

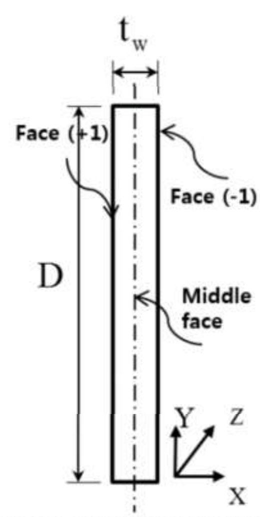

Figure 8. Section of web panels

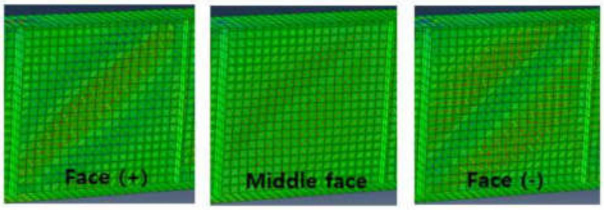

Figure 9. Principal stresses at failure point

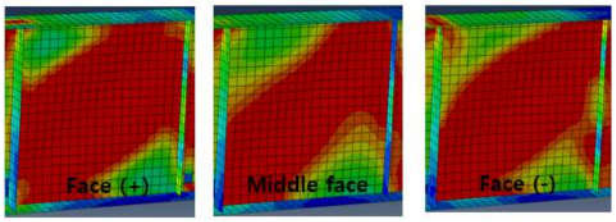

Figure 10. Von-Misses stresses at failure point

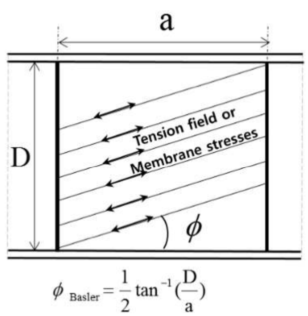

Figure 11. Basler's tension field in Basler's model

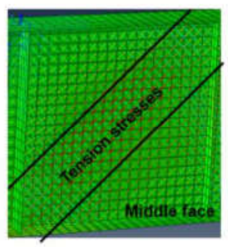

Figure 12. Tension stresses in Development post-buckling stage

2.3. Developing of post-buckling for aspect ratio =3

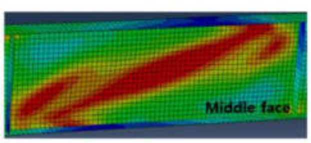

Figure 13. Von-Misses stresses at failure point for aspect ratio =3

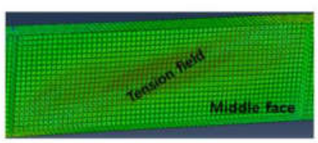

Figure 14. Tension stresses (membrane stresses) at failure point for aspect ratio =3

The current FEM results as Fig. 13 and 14 reveal that no tension field exists as the tension field which was assumed in all classical theories for the long web in Fig. 15. It means that Basler's tension field theory is totally invalid for long webs. In fact, Basler himself could not provide logical arguments that why is the slope of the tension field of $\phi_{Basler} = \frac{1}{2}\tan^{-1}(\frac{D}{a})$

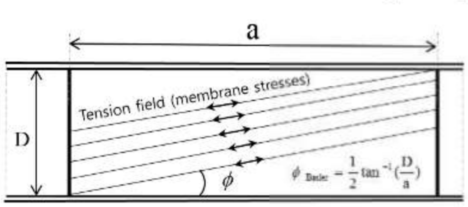

Figure 15. Basler's tension field in Basler's model for long web

The image in Fig. 16 extracted from the experimental study (1999) by Lee and Yoo, easily causes a misunderstanding that the red domain is the action of the tension field which was the premise of all classical tension field theories such as Basler's tension theory in Fig. 17. Unfortunately, from the current FEM results in Fig. 18 and 19, we can see that this domain is not tension field, but it includes both yielding compressive steel and yielding tensile steel.

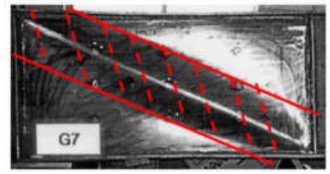

Figure 16. Image from Lee and Yoo's test in 1999

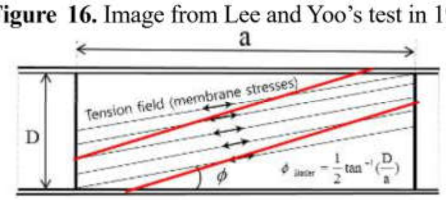

Figure 17. Basler's tension field in Basler's model for long web panels

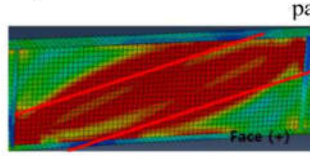

Figure. 18 Von-Misses stresses at failure point for aspect ratio =3

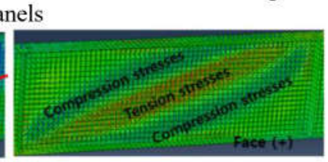

Figure. 19 Tension stresses (membrane stresses at failure point for aspect ratio =3)

3. CONTRIBUTION FROM FRAME ACTION OF A GIRDER

Accordingly, this part used the Model investigated in part 2 as Fig. 20 with various ratios of flange to web ($4 \le t_f / t_w \le 12$) and different slenderness ($200 \le D/t_w \le 800$). Then, it traces shear strength at the earliest yielding, O element to obtain shear strength when the initiation of yielding of web panels occurs. The FEM results were presented in Table 5.

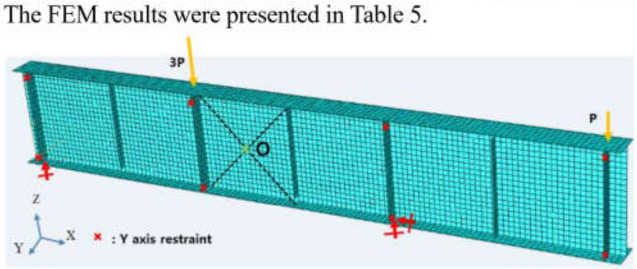

Figure 20. Boundary condition and applied load

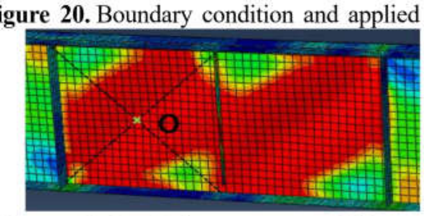

Figure 21. Von-Misses stresses at failure point

Table 3. Ultimate shear strength (kN) from the FEM results for girder with aspect ratio=1 and the heavy flanges

	Parameters of girders			
	D/t_w =200 T_f/t_w =4	D/t_w =200 T_f/t_w =6	D/t_w =400 T_f/t_w =8	D/t_w =800 T_f/t_w =12
Shear strength at yielding of element O (V_y) kN	2641.8	2712.2	2276.8	2022.2
Ultimate shear strength (V_U) kN	2938.7	3058.6	2883.8	2688.6
(V_U - V_y) kN	296.9	346.4	607.0	666.4
(V_U /V_y) kN	1.11	1.13	1.27	1.33

From Table 3, it can be seen that for the girder with slenderness ratio =200 and the ratio $t_f/t_w = 4$, the earliest yielding at element O occurs is at a shear strength of 2641.7 kN and then the girder continues to carry the load. Consequently, the failure of the girder is at the ultimate shear strength of 2938.7 kN. Obviously, at a shear strength of 2641.7 kN, the investigated web panel initiates to yield, thus this area of yielding steel become meaningless in the balancing mechanism. It means that the further increase in shear strength ($V_U - V_y$), it is 11% of V_y, is definitely not the contribution of web panels, but it could be the contribution of the frame action in the girder. The effect of this frame action was found first by Vierendeel and later in research related to post-buckling, several researchers mentioned this frame action as Chern and Ostapenko (1968) and White et al (2008). However, it has not quantified accurately the contribution of this frame action so far.

Noticeably, some of the models such as Cardiff's model (proposed by Porter et al. in 1975 and then known as Cardiff model) accounted for the explicit contribution from the flanges to the shear strength, but it is not truly Vierendeel frame action because this Vierendeel frame action is cooperating between the flanges and the vertical components as the transverse stiffeners. Therefore, Vierendeel frame action is affected by not only the dimension of the flanges but also the transverse stiffener spacing and the size of the transverse stiffener.

The following, table 3 reveals that shear strength $(V_U - V_y)$ rose with rising the ratio t_f/t_w and increase slenderness ratio. Even if for girder the very heavy flanges $t_f/t_w = 12$ and extremely high slenderness ratio of 800, the further increase in shear strength contribute from the Vierendeel frame action is 33% of V_y.

From above behavior for girder with the heavy flanges and high slenderness ratio, it goes to conclude that the measurement for ultimate shear strength of test girder with the heavy flange and high slenderness ratio can encompass the significant contribution from the Vierendeel frame action. However, the flange frame action needs to be considered a dependency between the shear strength and the applied moment, since the Vierendeel resistance of the flanges is influenced by the magnitude of the flange flexural stresses. Therefore, these test values can be not representative of web post-buckling behavior.

With respect to the comparison of Basler's results with test results for specimens with heavy flanges and high slenderness ratio, Basler's theoretical tension field yields corresponding to the failure of web panel in Basler's assumption. However, as in observed behaviors for the heavy-flange-girders, the ultimate shear strength obtained from these test could consist of further shear strength $(V_U - V_y)$ which increases after some elements of the web panel yield. In other words, these good correlations in Table 3 do not reflect the accuracy of Basler's theory.

4. CONCLUDING REMARKS

In case it accepts the existing paradox in Basler's theory, Basler's formula may still provide a reasonable prediction in a certain range of girder parameters.

However, Basler's formula gives serious predictions for square slender web panels with the moderate flanges

Basler's tension field theory is totally invalid for a long web panel because there is no tension field existing like the classical tension field for the long web panel. Baler's model, therefore, underestimates the ultimate shear strength for the long web panel.

Basler's predictions can correlate excellently with results from the different models which is contrary to Basler's model in the physical meaning. This indicates that the good correlation between the results calculated by any proposed models with test results is not trustworthy as the expectations of the prior researchers.

This behavior says that some of the test results correlating well with predictions by Basler's equation are merely contingent.

Due to the existing inconsistencies in Basler's theory, it is necessary to have serious considerations concerning the tension field theory. In general, the in-plane classical tension field cannot reflect accurately the developing of web post-buckling.

This study provides cautions for test results of specimens with the heavy flanges. These test results can consist of the remarkable contribution of the frame action in the girder model only subjected to shear. Therefore, the actual resistance of the frame action much less than those consisted of test values. This study with high web slenderness can be applied to aerodrome structures.

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