Journal of Engineering Science and Technology
Vol. 12, No. 11 (2017) 2839 - 2854

FINITE ELEMENT ANALYSIS OF TAPERED COMPOSITE PLATE GIRDER WITH A NON-LINEAR VARYING WEB DEPTH

Q. A. HASAN*, W. H. WAN BADARUZZAMAN,
AHMED W. AL-ZAND, AZRUL A. MUTALIB

Department of Civil and Structural Engineering,
The National University of Malaysia, 43600, UKM Bangi, Malaysia
*Corresponding Author: Qahtan.hasan@yahoo.com

Abstract

The paper presents Finite Element Analysis to determine the ultimate shear capacity of tapered composite plate girder. The effect of degree of taper on the ultimate shear capacity of tapered steel-concrete composite plate girder with a nonlinear varying web depth, effect of slenderness ratio on the ultimate shear capacity, and effect of flange stiffness on the ductility were considered as the parametric studies. Effect of concrete slab on the ultimate shear capacity of tapered plate girders was also considered and it was found to be so effective on the ultimate shear capacity of the tapered plate girder compared with the steel one. The accuracy of the finite element method is established by comparing the finite element with the results existing in the literature. The study was conducted using nonlinear finite element modelling with computer software LUSAS 14.7.

Keywords: Tapered composite plate girder; Tapered steel plate girder; Composite action in tapered girders, Ultimate shear capacity; Finite element method.

1. Introduction

When a hot-rolled section of I-beam is insufficient to carry the high load over long spans, built up plate girders are employed to satisfy the design requirement. However, the large dimensions of those built up plate girders could be undesirable in the economy and web slenderness limitations. Therefore, the taper in web panel could show improvement in performance, weight reduction, strength distribution, and aesthetic purpose of plate girders [1].

During the past few decades, considerable researches were done on behaviour of Prismatic plate girders using physical, theoretical methods. The diagonal tension

field and the equilibrium solution were ones of those methods in which have given a considerable attention. Basler and Thuerlimann [2] presented a plastic design method of web plate panels which allows for the influence of flange rigidity upon the post buckled behaviour of webs. Many researchers have follow Basler`s procedure to calculate for the ultimate shear capacity of straight and horizontally curved steel plate girders and even the ultimate shear capacity of tapered steel plate girders. Basler`s procedure has been developed to cover the composite action effect on the total stiffness of plate girders by Narayanan et al. [3]. They investigated the shear strengths of composite plate girders, with rectangular web cut-outs experimentally. The tests indicate that if adequate tension composite girder would be significantly higher than that of the plate girder alone.

Experimental and theoretical studies have been conducted on non-prismatic steel plate girder by researchers. It is observed from the previous literatures [4] that there is limited knowledge available in which focuses on the behaviour of non-prismatic plate girders, and none of those studies has gone through the behaviour of composite tapered plate girders. It was also shown that the current design code is based on prismatic plate girder Eurocode 3 [5]. Bedynek et al. [6] focused on both, critical shear load and ultimate shear resistance. Moreover, the post-buckling behaviour of tapered plates was studied. Some parametric studies with various geometries of tapered panels were done in order to find the most favourable design situations. The analysed parameters were: the panel aspect ratio, the inclined flange angle, the web and the flange slenderness. A study investigated the collapse mode of tapered plate girders loaded within the tip using theoretical expression based on the tension field theory [7]. The study presented the collapse mode of failure for tapered plate girders loaded within the tip. The theory provided an identical collapse load whether obtained from lower or upper bound solutions. Experimental collapse loads agree well with theory when the reduced plastic moment capacity of each flange is considered.

Falby and Lee [8] have developed the tension field formula for prismatic webs to be valid for the non-prismatic ones. Another study has been conducted to calculate for critical shear buckling stress of slender tapered web panels under shear loading and combined shear and bending moment. An analytical formulation is presented based on the common tension field method as well as numerical studies of plate instability, takes into account the influence of all the geometric design parameters and the presence of the flanges by [9-11]. The analytical method developed in the study is capable of predicting the ultimate shear capacity of tapered steel plate girders in a simple and precise manner. The study also included numerical studies and analytical formulation related to shear-bending moment interaction in tapered girders.

On the same bases of the previous study, the shear capacity and critical shear web buckling stress of tapered plate girders was evaluated by Zárate and Mirambell [12]. The Critical shear buckling load and the ultimate shear resistance of tapered plate girders were investigated with experimental and numerical analysis by considering the potential yielding of the tapered plates [13]. The numerical results showed that the critical shear buckling force of the inclined compression flange is higher than that of tension inclined flange. Bhurke and Alandkar [14] carried out a comparison between plate girder with uniform web depth and tapered web depth using FEM. The results of different parametric

studies show that the tapered web plate has same buckling strength however it achieves better economic performance.

This paper focuses on the effect of degree of taper on the ultimate shear capacity of steel-concrete composite plate girder bridges with nonlinear varying web depth (tapered plate girders) using nonlinear finite element modelling with computer software LUSAS 14.7. The F.E. modelling was verified and confirmed with the experimental results by Shanmugam and Baskar [15]. Degree of taper was considered as the parametric study. Effect of composite action on the tapered plate girder was also considered

2. Finite Element Modelling Technique and Geometrical Properties

This section describes the modelling technique and Geometrical properties of plate girders used in this study which are based on the tested plate girders by Shanmugam and Baskar [15]. Girder CPG1 was considered as the base model of the current study. Parametric studies were applied to the girder with the same material properties (yield stress fy for flanges, rebars and webs are 272 MPa, 300 MPa, and 286 MPa respectively, while cube compressive strength fcu of concrete is 40.2 MPa, modulus of elasticity of concrete and steel are 23.5 GPa and 209 GPa respectively) of the physical test by Shanmugam and Baskar [15]. The girders were modelled using the available technique in the finite element software LUSAS 14.7.

The overall depth of the girders is 790 mm whilst the thickness of the top and bottom flanges is 20 mm with 200 mm in width and the depth of the web is 750 mm with 3 mm thickness. The overall dimensions of the base plate girder CPG1 is illustrated in Table 1.

Table 1. Geometrical properties of the adopted plate girder.

Members dimension	Overall girder length (mm)	Web (mm)	Flanges (mm)	End post stiffeners (mm)	Transverse stiffeners (mm)
Length	3246	-	3246	400	-
Width	200	1141	200	200	200
Depth	790	750	-	-	750
Thick.	-	3	20	10	20

It was consider [15] that shear studs were designed to make full interaction between the top flange of the steel girder and the concrete slab. Based on that, the modelling of the interaction between the concrete and steel are assumed to be fully connected without considering the shear studs. In other words, the shear studs were replaced by full connection technique which is available LUSAS 14.7. Steel bars were modelled as a line with a given cross-sectional area and material properties. For the consideration of the assigning the geometrical properties, concrete and reinforcements are defined as volume and line respectively. Details of the concrete slab are shown in Fig. 1.

Two layers of welded mesh (10 mm diameter bars spaced at 150 mm centre to centre in orthogonal directions) were used along the entire length of the concrete slab.

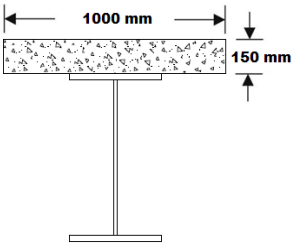


Fig. 1. Cross-section of the composite plate girder

The changing in depth of web panel is varied nonlinearly and the bottom flange is shaped parabolically in order to be connected to the lower edge of the web panel. Reduction in the web panel at the mid-span (d2) is given in Table 2. The depth of web panel at the mid-span (d1) is reduced 25 times (in each time about 15 mm) in order to reduce the depth of web panel to the half, while the edges at the supports (d) are kept with the same depth. The selected reduction is chosen randomly to study the effect of degree of taper until the depth of web panels is reduced to the half (d/2). The degree of taper β is listed in Table 2, and it is calculated based on the geometry of the plate girder in Fig. 2.

The modelling technique of using the line of symmetry which is available in LUSAS 14.7 was applied on a half model of the plate girders used in the current study as shown in Fig. 2. In this computer code the element stiffness matrix is continuously updated by the Newton–Raphson iterative procedure with the material nonlinearity accounted for. Incrementation of the applied load was set to be automatic and the start load factor was inserted as 0.1 kN in order to avoid the crack points which usually happen between elements when it is subjected to high sudden applied load. The load was applied in 6 points at top and bottom surfaces on the concrete slab at the mid span with an increment step of 0.5 KN, therefore, the total load factor per increment is 3 kN. The geometrical nonlinearity in the nonlinearity analysis options was chosen to be solved with total Lagrangian. In the elastic range, the values of Young's modulus and Poisson's ratio were used as 209 GPa and 0.3 respectively. In the inelastic range, Von Mises yield criteria was used to define isotropic yielding of the steel girder. Modified Von Mises yield criteria were used to define isotropic yielding of the concrete slab in order to insert the initial compressive and tensile strength of the concrete. The tensile strength of the concrete was assumed to be about 10 % of its compressive strength. A typical finite element model of a steel-concrete composite plate girder is shown in Fig. 3.

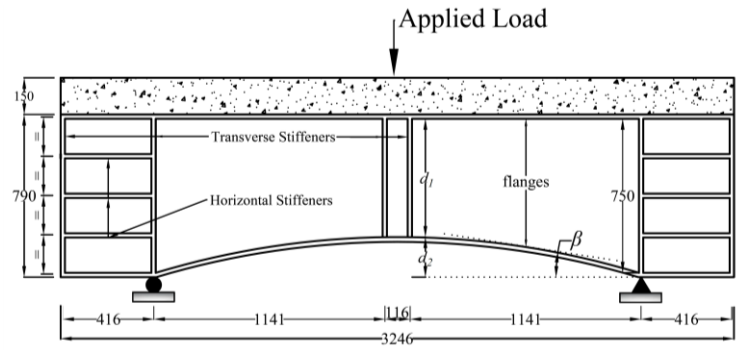


Fig. 2. Details of typical tapered composite plate girder.

Table 2. Properties of the taper in plate girders.

No.	Girder	Reduction in web depth at mid-span (d_2)	Depth of web penal d_o	Degree of Taper β^o
1	**CPG1**	0	750	0.00
2	**CTPG1**	15	735	0.76
3	**CTPG2**	30	720	1.53
4	**CTPG3**	45	705	2.29
5	**CTPG4**	60	690	3.05
6	**CTPG5**	75	675	3.81
7	**CTPG6**	90	660	4.57
8	**CTPG7**	105	645	5.33
9	**CTPG8**	120	630	6.09
10	**CTPG9**	135	615	6.84
11	**CTPG10**	150	600	7.59
12	**CTPG11**	165	585	8.34
13	**CTPG12**	180	570	9.09
14	**CTPG13**	195	555	9.83
15	**CTPG14**	210	540	10.57
16	**CTPG15**	225	525	11.31
17	**CTPG16**	240	510	12.04
18	**CTPG17**	255	495	12.77
19	**CTPG18**	270	480	13.50
20	**CTPG19**	285	465	14.22
21	**CTPG20**	300	450	14.93
22	**CTPG21**	315	435	15.64
23	**CTPG22**	330	420	16.35
24	**CTPG23**	345	405	17.05
25	**CTPG24**	360	390	17.74
26	**CTPG25**	375	375	18.43

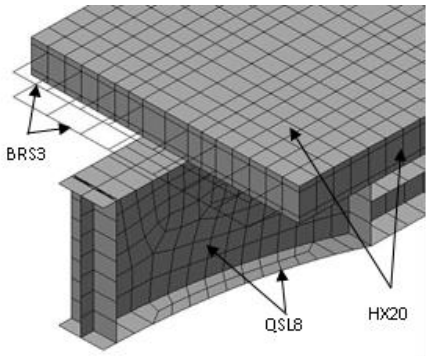


Fig. 3. Typical finite-element model of tapered steel-concrete composite plate girder.

3. Mesh and Validation of the modelling technique

In finite element analysis of solving civil engineering problems, it is desirable to choose mesh type in which can show better results in a reasonable time. Different

types of mesh can show the same accuracy of a specific problem, however, time to get the results of each mesh is different depending on the number of nods for each type of element. In this study, line, surface, and volume mesh are used with specified element types of each mesh category. The element types were checked in each case in order to find a better accuracy in short time of running the calculation of the finite element analysis. The following types of mesh were found to show better accuracy compared with experimental results taken into account the time consuming. Line mesh element type (BRS3) was used for the reinforcements. Irregular quadratic thin shell element (QSL8) and triangular thin shell element (TSL6) were chosen for the surface mesh (steel girder), and solid element (HX20) was used for the volume mesh (concrete slab) were adopted in the analysis. Details and properties of the chosen elements are illustrated in Table 3. Figure 3 shows the typical mesh used in the analysis.

Table 3. Details and properties of the selected elements.

Element Name	Element Group	Element Subgroup	Number of Nodes	DOF at nods	Material properties	Materials non-linearity
BRS3	Bars	Structural Bars	3	3	Isotropic	Linear
TSL6	Shells	Semiloof Shells	6	5	Isotropic	Non-linear
QSL8	Shells	Semiloof Shells	8	5	Isotropic	Non-linear
HX20	3D Continuum	Solid Continuum	15	3 at each nod		Non-linear

Convergence study has been conducted to determine the optimal mesh; different element sizes were chosen, 200, 150, 100, 80, 65, and 50 and the total given elements by the selected sizes were (1225, 1315, 1800, 1993, 2588, and 6184) respectively. It was found that the ultimate shear load capacity of the composite plate girder was reduced by reducing the element size. However, it became almost constant starting from element size of 80 and less as shown in Fig. 4. Based on that, element size 80 was, therefore, considered as an optimal mesh size in which can give a better performance with a considerable time reduction. It was also noted that using small element size is no longer useful, since, it gives the same accuracy with longer time of solving with the finite element method.

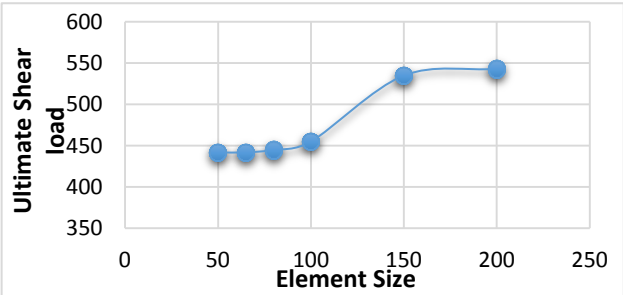


Fig. 4. Convergence study of finite element models.

The validation of the finite element models were verified with the test result of CPG1 by comparing the load-displacement curve as shown in Fig. 5. The comparison showed good agreement between the experimental and finite element results in terms of ultimate shear capacity. The ultimate shear obtained by FEA for the prismatic girder was 873 kN, which showed an increasing of about 12 kN in the ultimate shear capacity when compared with the experimental result of CPG1. However, a slight difference was found in the load-displacement behaviour cure of the experimental and finite element analysis to an acceptable level. It is justified that the assumed full connection between the top flange and concrete slab by finite element analysis gives a slight different behaviour from that of real behaviour of girders with shear studs and hence increase the stiffness of plate girder.

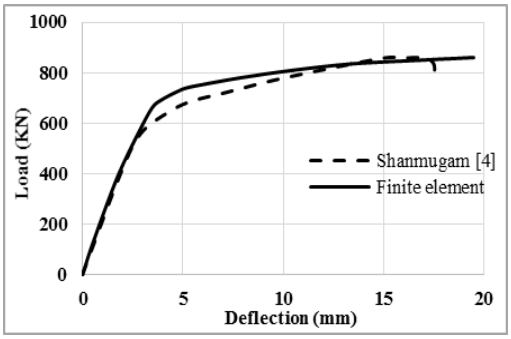
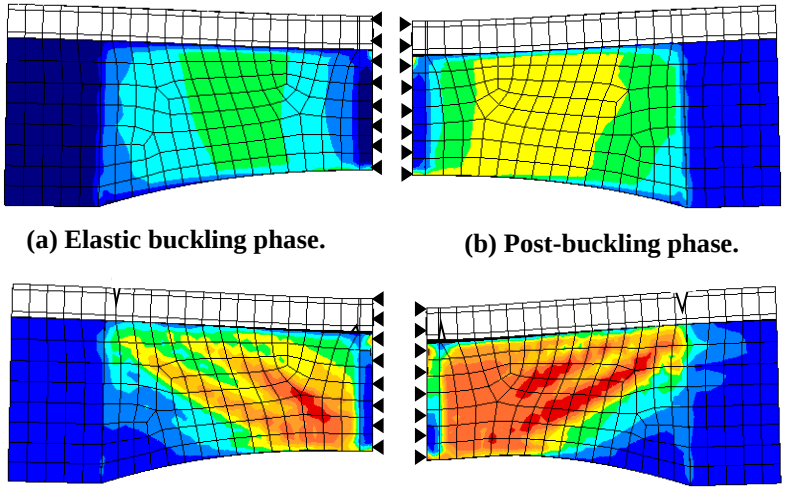


Fig. 5. Validity of the finite element analysis.

4. Collapse behaviour of composite tapered plate girders

The presence of the concrete slab increases the total stiffness of the plate girders. It is well known that failure mechanism of prismatic plate girders occurs in four phases. Firstly, local web buckling occurs when the girder loaded up to the critical shear stress and the additional load is carried by a tensile membrane stress develops diagonally within the web panel. This tensile membrane stress is anchored with the transverse stiffeners and the flanges [7]. As the load goes further, the web yields under the effect of the combination of the tensile membrane stress and buckling stress. After the yield of the web panel, collapse of the girder occurs with the formation of four plastic hinges in the top and bottom flanges followed by extensive cracking in the concrete slab. The spread of the membrane stresses on the compression flange seems to be wider than that of bottom flange due to the strong anchorage caused by the composite action between the compression flange and the concrete slab which gives rise to stronger flange action in compression compared to tension side [15].

From the point of view of FE analysis, it can be said that the collapse behaviour of non-prismatic composite girders seems to be the same with the prismatic ones as shown in Fig. 6. The first phase is shown in Fig. 6(a) in which shows the local buckling in the light green and light blue colours within the web panel and small deflection occurs and the web plate still in the elastic range. The development of the web buckling as the load keep increasing becomes obvious (post buckling) and it is remarked in the yellow and light green colours as shown in Fig. 6(b) by the time that the deflection at the mid-span increased.



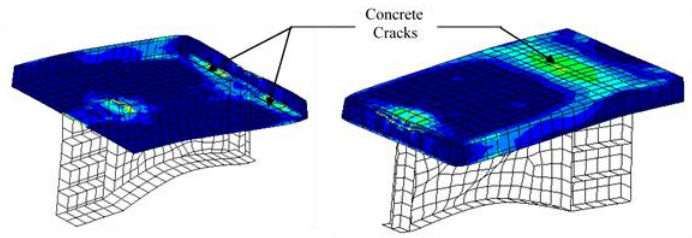
(a) Elastic buckling phase. **(b) Post-buckling phase.**

(c) Development of plastic hinges in the flanges. **(d) Shear crack in the concrete slab.**

Fig.6. Four phases of load carrying mechanism in a web panel with opening of a composite plate girder.

When the web panel yields and the spread of the membrane stress takes its final place at the top and bottom flanges as shown in Fig. 6(c), the load is carried by the flange and the concrete slab. In this stage, the interaction between the concrete slab and the top flange plays key role in increasing the stiffness of the top flange and then the total stiffness of the composite tapered girders [16]. Finally, four hinges form at top and bottom flanges and concrete slab starts to get cracked. Figure 6(d) shows that the spread of membrane stress on the top flange is wider than that on the bottom flange due to the composite action. Final collapse occurs when the cracks of concrete is developed and become larger by the time of the collapse of the steel girder with the development of the four hinges.

The concrete cracks developments start at the third phase when the deformation of plastic hinges on the top and bottom flanges starts to appear. The collapse of the steel plate by the plastic hinges is followed by the crush of the concrete at a location above the support and at the mid-span as shown in Fig. 7.



(a) Concrete crack at the mid-span. (b) Concrete crack at the support.

Fig. 7. Typical FE analysis contour plots of concrete cracks at the mid-span and the support.

5. Influence of Degree of Taper on the Ultimate Shear Capacity

A number of composite tapered plate girder models were analysed using finite element method in order to examine the effect of degree of taper on the ultimate shear capacity. Drop in the ultimate shear capacity, drop in weight of web panel, and effect of composite action are investigated and illustrated in Table 4. The table also shows the drop in weight and ultimate shear capacity of the tapered webs.

The shear capacity of girder with constant web depth (prismatic web) was found to be 873 kN. The corresponding values of the ultimate shear capacities for girders CTPG (5-25) are 808 kN, 746 kN, 680 kN, 616 kN, and 550 kN respectively. The maximum drop in the ultimate shear capacity was occurred when the maximum degree of tapered (0.5d represented by CTPG25) is selected. Where d is the depth of the web panel. From Table 4, the magnitude of the maximum drop in shear capacity occurred between girders CTPG1 (873 kN) and CTPG25 (550 kN) is about 36%. It is then, due to the reduction in the web panel area and the spread of the diagonal tension field on the bottom flange which seems to be differ from the tension field action in non-tapered webs.

Table 4. Predicted results showing the effects of degree of taper on the shear capacity.

No.	Girder	Drop in weight of web panel %	Ultimate Capacity (kN)	Drop in the ultimate capacity %
1	**CPG1**	0	873	0
2	**CTPG1**	2.68	859	3.04
3	**CTPG2**	3.43	859	3.04
4	**CTPG3**	4.62	859	2.93
5	**CTPG4**	5.91	840	5.19
6	**CTPG5**	7.25	808	8.80
7	**CTPG6**	8.60	794	10.38
8	**CTPG7**	9.98	784	11.51
9	**CTPG8**	11.36	772	12.86
10	**CTPG9**	12.75	760	14.22
11	**CTPG10**	14.14	746	15.80
12	**CTPG11**	15.54	734	17.15
13	**CTPG12**	16.95	718	18.96
14	**CTPG13**	18.36	720	18.73
15	**CTPG14**	19.77	694	21.67
16	**CTPG15**	21.19	680	23.25
17	**CTPG16**	22.61	670	24.37
18	**CTPG17**	24.04	658	25.73
19	**CTPG18**	25.48	644	27.31
20	**CTPG19**	26.92	630	28.89
21	**CTPG20**	28.36	616	30.47
22	**CTPG21**	29.81	600	32.50
23	**CTPG22**	31.27	588	33.63
24	**CTPG23**	32.73	574	35.21
25	**CTPG24**	34.20	562	36.56
26	**CTPG25**	35.67	550	37.92

where *h* is the depth of the prismatic and non-prismatic plate girders, and h1 is the depth of shortened side of tapered web panel.

It is observed from Table 4 that the drop in the ultimate shear capacity is about 36% due to the maximum selected degree of taper represented by girder CTPG25. It is also noted that the changing in the ultimate shear capacities dropped in a linear manner as illustrated in Fig. 8. Relation between ultimate shear capacity and the degree of taper dropped almost linearly due to the drop in the web panel weight. It can also be observed from the figure that the drop in shear strength is almost the same drop in the weight of web panel.

Additional investigation has been conducted on steel tapered plate girders STPG (5-25). The ultimate shear capacities of those girders are compared with the capacities of composite tapered plate girders CTPG (5 to 25) as illustrated in Fig. 9. It is observed from the figure that the drop in the ultimate shear capacity of steel tapered plate girders due to the increase in the degree of taper seems to be less than those of composite tapered plate girders and the line appears to be more flatten which has less degree of inclination.

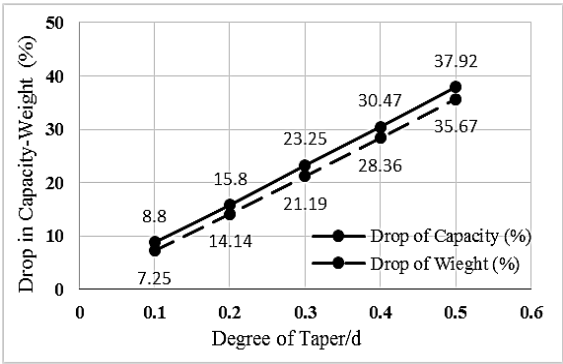


Fig. 8. Drop in capacity-weight with different degree of taper.

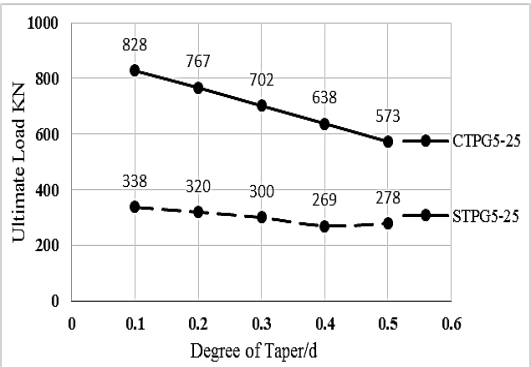


Fig. 9. Ultimate capacities of composite and steel tapered plate girders.

6. Influence Of Concrete Slab on The Ultimate Shear Capacity

It was pointed out [3] that when an adequate connection is provided between the top flange of steel plate girder and concrete slab, the ultimate shear capacity of the composite girder is significantly higher than that of steel plate girder alone. Based on the study by Shanmugam and Baskar [15] the increase in the ultimate shear capacity of composite plate girder when compared with steel plate girder was about 43 % for prismatic girders (CPG1, SPG1). On the same bases, steel and composite tapered plate girders (STPG5-25, CTPG5-25) in which each pair (i.e. STPG5 and CTPG5) has the same dimensions of the steel plate were compared in terms of their ultimate shear capacities and a significant average increase about 56 % can be seen from Fig. 10. This increase could be due to the enhancement of the anchorage between top flange and the concrete slab, and the additional strength provided by the slab. Finite element analysis showed that the ultimate shear capacity of STPG(5-25) are 338 kN, 320 kN, 300 kN, 269 kN, and 278 kN and CTPG(5-25) are 828 kN, 767 kN, 702 kN, 638 kN, and 573 kN respectively as shown in Fig. 8. It can be seen from the figure that the drop in the ultimate shear capacity for composite tapered girders is higher than that of steel plate girders. It also can be seen that the ductility didn't seem to be affected by the added the concrete slab compared to the steel one.

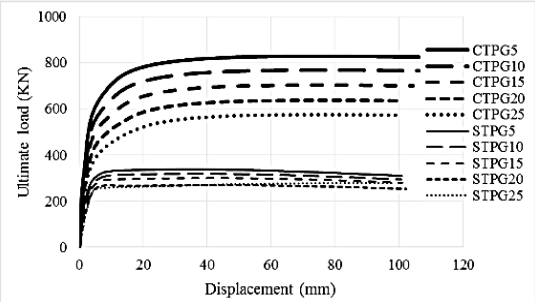


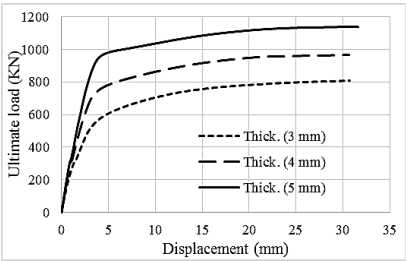
Fig. 10. Load-displacement curve of steel and composite tapered girders.

7. Influence of Web Slenderness on the Ultimate Shear Capacity

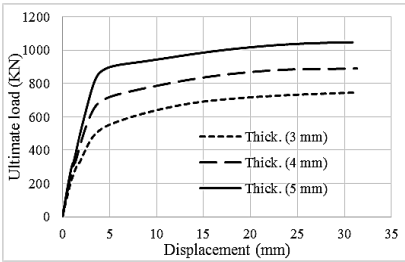
In order to investigate the influence of web slenderness on the ultimate shear capacity of tapered composite plate girders, the web area of the tapered steel girders was kept constant and only the web thickness was changed. This web thickness can affect the web slenderness ratio in which can directly influence the ultimate shear capacity, since it is considered that the shear loading is carried by the web alone. It was stated [17] that in normal practice, plate girders are designed with web slenderness ratio ranging from 120 to 160 and it is allowed that the maximum web slenderness ratio is up to 250. Based on the above factors, three different web slenderness ratios, namely 150, 187.5, and 250, with web plate thickness 5, 4, and 3 respectively, were considered in this study in order to evaluate their effectiveness on the ultimate shear capacity. The panel aspect ratio of the web was kept as 1.5 for all girders. The chosen web depths were applied on the girders CTPG5, CTPG10, CTPG15, CTPG20, and CTPG25.

A significant raise of resistance of the tapered composite girders can be drawn for each increment of the web thickness. It can be seen from the load-

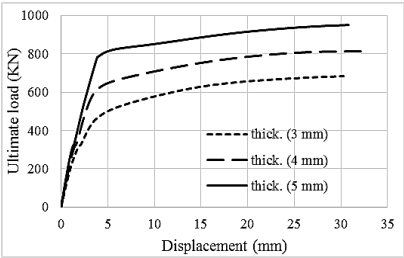
displacement curves in Fig. 11 that the ultimate shear capacity of each girder has been increased significantly as the thickness of the web was increased. Figure 11(a) shows that the ultimate shear dropped considerably as the web slenderness was increased. As an example, the ultimate shear capacity of CTPG15 with web thickness 3 mm was increased from 681 kN to 812 kN as the web thickness was increased to 4 mm, an improvement of 19.2%. Also, using web thickness 5 mm for the same plate girder leaded to the ultimate shear capacity of 950 kN which is 17% higher than that of using 4mm web thickness. Therefore, the use of web slenderness ratio 150 seemed to be more effective on the ultimate shear capacity of tapered plate girders than slenderness ratios of 187.5 and 250. A 39.5% was the total increase in the ultimate shear loading due to the increase in web plate thickness from 3 mm to 5 mm as illustrated in Table 5.



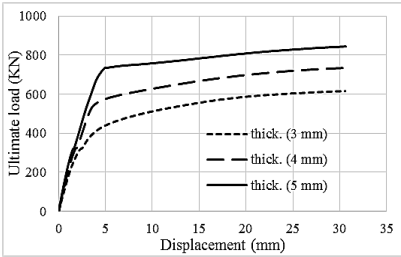
(a) Load-displacement curve of CTPG5.



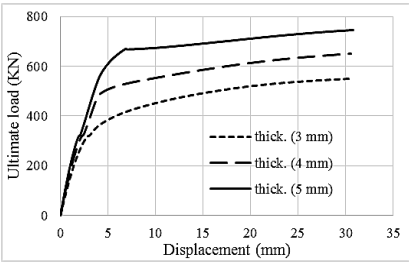
(b) Load-displacement curve of CTPG10



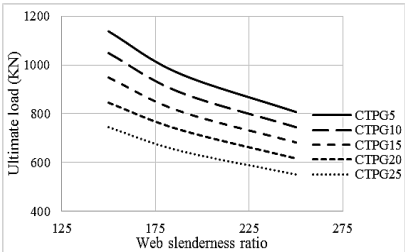
(c) Load-displacement curve of CTPG15.



(d) Load-displacement curve of CTPG20.



(e) Load-displacement curve of CTPG25.



(f) Load-slenderness ratio of different girders.

Fig. 11. Influence of web slenderness ratio on the ultimate load capacity.

Table 5. Influence of d/t on the ultimate shear capacity.

No.	Girder	Slender-ness ratio	Web thickness (mm)	Ultimate shear (kN)	% increase in the ultimate shear
		250	3	807	0.00
1.	**CTPG5**	187.5	4	966	16.46
		150	5	1138	29.08
		250	3	746	0.00
2.	**CTPG10**	187.5	4	890	16.18
		150	5	1048	28.81
		250	3	681	0.00
3.	**CTPG15**	187.5	4	812	16.13
		150	5	950	28.31
		250	3	615	0.00
4.	**CTPG20**	187.5	4	735	16.32
		150	5	845	27.21
		250	3	551	0.00
5.	**CTPG25**	187.5	4	650	15.23
		150	5	746	26.13

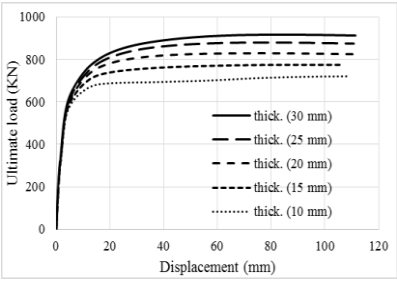
8. Influence of Flange Stiffness on the Ultimate Shear Capacity and Ductility

When tension field action occurs in a plate girder, as was specified in [17] then, the shear carrying capacity is calculated from the critical buckling strength of web, post buckling reserve strength of web panel and the yielding of flanges. However, the composite action between the compression flange and the concrete slab needs to be considered when calculating for the shear carrying capacity of composite plate girders. Flange sections are designed based on the beam theory at the primary design. Such a design provides a minimum flange dimension which leads to the ultimate shear in perfect cases and lateral torsion mode of failure in some other cases. Therefore, plate girders are designed in such a way that they will not fail in lateral torsional buckling mode.

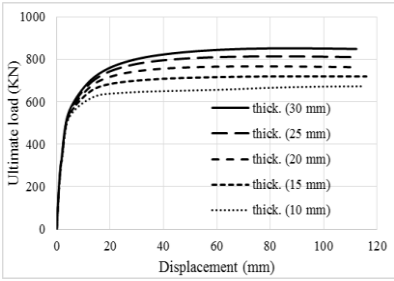
In this section, the flange thickness was chosen to be varied, while the other parameters were kept constant. The girders were investigated from the minimum flange thickness 10 mm which is required from the beam theory to a maximum limit of 30mm. Girders CTPG(5-25) with flange thickness of 10mm reached the ultimate shear of 721 kN, 673 kN, 614 kN, 555 kN, and 501 kN respectively as illustrated in Fig. 12. While the same girders with maximum flange thickness 30 mm reached the ultimate shear of 912 kN, 848 kN, 777 kN, 705 kN, and 631 kN respectively as illustrated in Fig. 12. A considerable average increase in the ultimate shear capacity of composite tapered plate girder (20%) has been noticed as the flange thickness was increased. It is proved that the minimum thickness of flanges could reach the ultimate shear but no ductility would be changed.

Based on the finite element analysis, further increase in flange thickness did not show any significant improvement in the ductility of composite tapered plate girders (Fig. 12). Since, the stiffness and ductility of such girders are almost governed by the composite action provided by the concrete slab and steel plate and any change in the flange thickness will not influence the ductility of composite tapered plate girders.

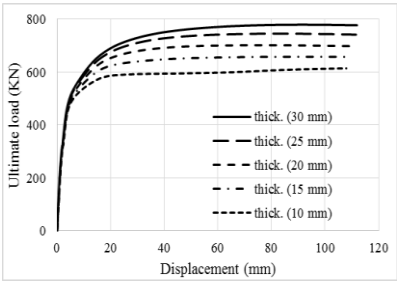
On the other hand, steel plate girder SPG1 by Baskar and Shanmugam [18] with various flange thickness (10-30 mm) has been analysed numerically. It is observed from Fig. 13 that the ultimate shear increased from 450 kN for flange thickness of 10 mm to 585 kN for flange thickness of 30 mm. It can also be noted that the increase in flange stiffness widens the curve and becomes more flatten with considerable increase in the deflection range. It reveals by the FE analysis that the ductility factor for steel plate girders increases i.e. it undergoes large deformations without decrease in the load, while, it showed a slight increase in the ductility behaviour of composite plate girders as illustrated in Fig. 13.



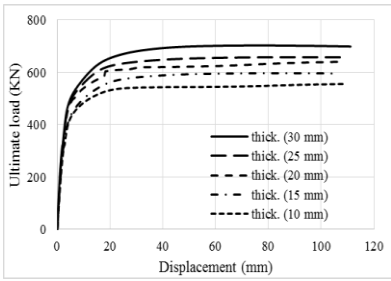
(a) Load-displacement curve of CTPG5.



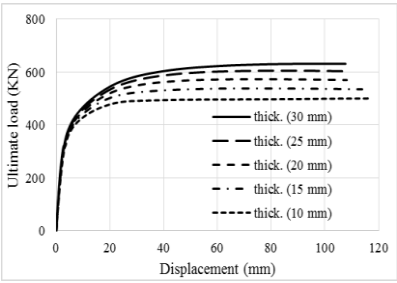
(b) Load-displacement curve of CTPG10.



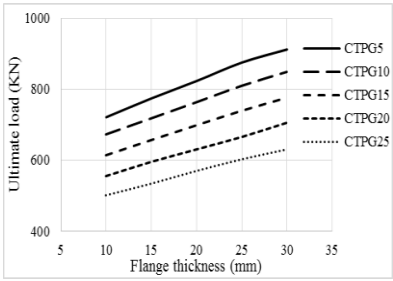
(c) Load-displacement curve of CTPG15.



(d) Load-displacement curve of CTPG20.



(e) Load-displacement curve of CTPG25.



(a) Load-flange thickness of different girders.

Fig. 12. Influence of flange stiffness on the ultimate shear capacity.

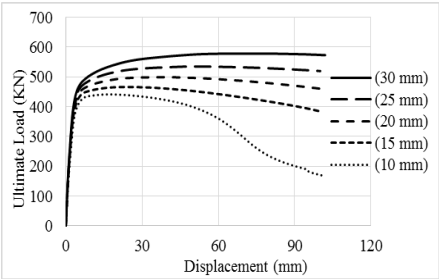


Fig. 13. Load-displacement curve of steel plate girders with various flange thickness.

9. Conclusions

Finite element models to predict the ultimate shear capacity of tapered composite plate girders with different degree of taper are presented in this paper. A number of steel and composite tapered plate girders with different degree of taper, web slenderness ratios, flange stiffness and influence of concrete slab were analysed using the finite element method (LUSAS 14.7). Significant conclusions can be drawn from this paper as below.

- The collapse mode of composite tapered girders seems to be the same with the composite non-tapered girders as it was found from the current finite element collapse mode.
- The ultimate capacity of tapered plate girders is improved about 56 % due to full composite action provided by the anchorage between the reinforced concrete slab and the compression flange.
- A significant raise of resistance of the tapered composite girders can be noticed for each increment of the web thickness. A 39.5% was the total increase in the ultimate shear capacity due to the increase in web plate thickness from 3 mm to 5 mm for girder CTPG15. However, the increase in the web slenderness ratio does not affect the ductility of composite tapered plate girders.
- The maximum influence of flange stiffness using flange thickness of 30 mm on the ultimate capacity of composite tapered plate girders is 20%. However, further increase in flange thickness does not show any significant improvement in the ductility of composite tapered plate girders. While, the ductility factor for steel plate girders increases i.e. it undergoes large deformations without decrease in the load with considerable increase in the deflection range.
- The corresponding finite element results show that the ultimate shear capacity is influenced significantly with the degree of taper, nearly 36% drop when the degree of taper is increased to maximum value which is represented by girder CTPG25. It was also found that the drop in the ultimate shear capacities is accompanied by almost the same values of the drop in weight of the web panels due to the degree of taper.

References

1. Hasan, Q.A.; Badaruzzaman, W.H.W.; Al-Zand, A.W.; and Mutalib, A.A. (2015). The state of the art of steel and steel-composite plate girder bridges.

Part I: Straight plate girders. *Thin-Walled Structures*. http://dx.doi.org/10. 1016 /j.tws.2015.01.014i

2. Basler, K. (1961). Strength of plate girders in shear. *Journal of Structural Division*, 87(7), 151-80.
3. Narayanan, R.; Al-Amery, R.; and Roberts, T. (1989). Shear strength of composite plate girders with rectangular web cut-outs. *Journal of constructional steel research*, 12(2), 151-166.
4. Galambos, T.V. (1998). *Guide to stability design criteria for metal structures*. John Wiley & Sons.
5. Eurocode 3: (2006). Design of steel structures, Part 1–5: General rules, Supplementary rules for planar plated structures without transverse loading, *ENV* 1993-1-5:2006 E.
6. Bedynek, A.; Real E.; and Mirambell E. (2013). Tapered plate girders under shear: Tests and numerical research. *Engineering Structures*, 46, 350-358.
7. Davies, G.; and Mandal, S. (1979). The Collapse Behaviour of Tapered Plate Girders Loaded Within the Tip. *in Ice Proceedings*, *Thomas Telford*.
8. Falby, W.; and Lee, G. (1976). Tension-field design of tapered webs. *Engineering Journal*, 13(1).
9. Shanmugam, N.; and Min H. (2007). Ultimate load behaviour of tapered steel plate girders. *Steel and Composite Structures*, 7(6), 469-486.
10. Mirambell, E.; and Zárate, A.V. (2005). Ultimate Strength of Tapered Steel Plate Girders Uuder Combined Shear and Bending Moment. *Journal of Advances in Steel Structures*, 2, 1383-1388.
11. Mirambell, E.; and Zárate, A.V. (2000). Web buckling of tapered plate girders. *Proceedings of the ICE-Structures and Buildings*, 140(1), 51-60.
12. Zárate, A.V.; and Mirambell, E. (2004). Shear strength of tapered steel plate girders. *Proceedings of the ICE-Structures and Buildings*, 157(5), 343-354.
13. Saladrigas, E.; Bedynek, A.; and Mirambell, E. (2011). Numerical and experimental research in tapered steel plate girders subjected to shear. *SDSS'Rio*.
14. Bhurke K.N.; and Alandkar, P.M. (2013). Strength of Welded Plate Girder with Tapered Web. *Int. Journal of Engineering Research and Application*, 3(5), 1947-1951.
15. Shanmugam, N.; and Baskar, K. (2003). Steel-concrete composite plate girders subject to shear loading. *Journal of Structural Engineering*, 129(9), 1230-1242.
16. Yatim, M.; Shanmugam, N.; and Wan Badaruzzaman W. (2013). Behaviour of partially connected composite plate girders containing web openings. *Thin-Walled Structures*, 72, 102-112.
17. BS 5950: Part 1: (1990). Structural use of steelwork in building. British Standard Code of Practice for Design in Simple and Continuous Construction: Hot Rolled Sections. *British Standards Institution*,.
18. Baskar, K.; and Shanmugam, N. (2003). Steel-concrete composite plate girders subject to combined shear and bending. *Journal of Constructional Steel Research*, 59(4), 531-557.