Experimental Study on Shear Strength Behaviours of the Composite UHPC Steel Girders in Slender web

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Abstract. Modern structures require light elements with high stiffness to weight ratios. Composite structures with high-performance material are particularly useful for long spans such as in bridges and large buildings. Conventional concrete is brittle, with very low tensile and high compressive strength; ultra-high performance concrete UHPC can be used to improve the ductility and strength of the deck slab, yet though many researchers have examined the behaviour of UHPC individually, this material requires investigation in conjunction with other variables. A composite structure of steel plate girders and concrete deck is thus examined in the present study, with seven plate girders and three composite girders. Flat web plate girders without stiffeners were investigated to estimate ultimate strength in two parameters, web slenderness, and flange slenderness. In addition, three composite girders were tested with web slenderness ratings 167, 250, and 375. The results show the effects of slenderness on shear strength and the failure accrued due to tension field action, while elastic buckling was seen to contribute to shear strength.

1. Introduction

Steel-concrete composite beams are used extensively in building and bridges. The composite sections for flexural members usually consist of concrete deck slabs and steel beams, which are classified into two types according to their manufacturing method into rolled shaped beams or built-up sections [1]. In this study, the behaviour of plate steel with built-up profiles was investigated experimentally. All plate girders were formed by welding two plates as flanges with a web plate to make an I-section [2]. A plate girder is a form of a deep beam, so the limit states of such beams are also applicable, and the nominal bending moment (Mn), is related to slenderness ratio λ, the lateral-torsional buckling, flange local buckling, and web local buckling. This study thus examined doubly and singly symmetric I-shaped welded plate girders with slender webs such that the web slenderness ratio λ satisfied Equation (1):

\[ \lambda = \frac{h}{t_w} > 5.7 \sqrt{\frac{E}{F_y}} \] (1)

The web carries most of the shear stresses, an as the plate girder has a thin web, stability is the first concern, along with elastic buckling and inelastic buckling when using stiffeners [1].

A brace perpendicular to the vertical axis may buckle laterally, though lateral supports at adequate frequent intervals will prevent the plate from lateral-torsional buckling [3]. While the spacing between the various points of lateral or torsional bracing further increases the limits, the section will have insufficient rotation capacity to permit full moment redistribution and thus will not permit plastic analysis [4], generating inelastic buckling. The panel formed between the flanges and stiffener is thus considered as a plate for the case of pure shear and simply supported edges, as shown in Figure 1.
The tension-field action, neglecting out-of-plane bending stress effects, offers acceptable approximations for plate girders with aspect ratios of \((a/D) \leq 1.5\), where \(a\) is transverse stiffener spacing, and \(D\) is girder depth [5]. Tension field theory is valid only if the inner web panel is supported with rigid stringers such as flanges and transverse stiffeners [6]. The web undergoes a state of elastic buckling while under increasing loads, developing permanent folds. After this buckled state develops, the web panel is still capable of carrying additional load due to the tension bands of loads, located at a diagonal state without compression loads [7]. Generally, the web panel is subjected to a combination of pure shear and diagonal tension, and the result of these two effects is an incomplete diagonal tension, as shown in Figure 1.

![Figure 1. Incomplete Diagonal Tension Principal [7].](image)

A composite action can be achieved by using mechanical connectors such as studs to resist longitudinal slip and vertical separation between concrete slabs and steel beams [8]. AISC commentary G2 neglects the effect of bending on the web, as plate girders usually use a slender web. In stockier webs, no bend-buckling may occur, but high web shear in combination with bending may cause yielding of the web adjacent to the flange. Plastic analysis is thus used where instability of the web is precluded [9]. Composite beams are usually laterally supported, with four parameters being used to demonstrate the impact on the strength of the beam.

2. Literature review

Wagner (1931), improved diagonal shear theory to assess the post-buckling shear strength of thin panels used in aircraft structures, though the post-buckling design was not used generally until 1960 in civil structures [10]. Basler first adopted post-buckling strength into AISC [1] specifications, while AASHO [11] followed suit, and the Cardiff model by Porter was adopted in British standard BS 5400 [5].

2.1. Flat web girders

Bergfelt [2] performed an experimental test to ascertain the influence of web stiffeners on thin-plate girders’ behaviours, revealing that the use of stiffeners enhanced the resistance of the beam, especially in the upper limits. Elgaaly [12] found that the presence of shear reduced the ultimate edge carrying capacity of the web, and developed a formula for the relationship between shear load and patch load. Post-buckling increases when the slenderness ratio of the web increases. Herzog [13] adopted a simple mathematical method to predict the ultimate shear strength of thin-walled plain steel plate and composite girders, whether such girders were stiffened or unstiffened (transversally and longitudinally). The research proposed a resistance factor of 0.8 for ultimate pure shear strength, 0.75 for combined shear and bending, and 1.11 for the composite section, observing that the arrangement of transfer stiffeners, unlike that of intermediate stiffeners, does not affect the final strength [13]. Lee and Yoo [5] revealed discrepancies of the failure mode between the assumed failure mechanism and
numerical analysis, as well as studying the boundary conditions of the flange-web juncture, and out plane bending. They also studied the effects of the relative thickness of the flange and web on buckling and post-buckling of the web panel, observed that the flange-web juncture is very close to fixity. The diagonal tension thus does not need support from a flange to accrue, and the ultimate shear capacity is influenced by the thickness of the flange such that increases in the thickness of flange increase the shear resistance of the web due to the supported behaviour of the flange.

The initial deflection effect on the collapse strength of a thin rectangular plate subjected to uniaxial compression was studied by Sadowsky and Chen [14, 8]. Plates alternatively normalised by amplitude to thickness ratio were measured. The lowest capacities due to initial deflection were obtained for single buckling shapes applied at between 11 and 21% of the higher values, depending on the plate slenderness. White et al. [15] compared numerical predicted equations for the shear strength capacity of plate I-girder sections with experimental results. The first variable studied was the coefficient of shear buckling. The second variable was the combination of shear strength post-buckling of the web with the flanges’ frame action. The results revealed that Basler models, which are simple expressions used in the AISC and AASHTO specifications, offered the best accuracy for calculating the shear resistance capacity of transverse stiffeners in I-Girders.

Roberts and Shahabian [16] proposed an interaction formula to compute the ultimate resistance of slender plate girder web panels based on the contributions of bending, shear, and patch loading. The interaction equation offered a reasonable correlation compared with experimental results. Daley et al. [17] found that the shear stress buckled linearly to 50 to 80% of ultimate stress, and the out of plane displacement was visible, becoming clearer at failure. The comparison showed that the slenderness ratio, D/tw, influenced the results according to the equation considered.

2.2. Composite Sections

Allison et al. [18] investigated the interactions between tension field action and shear connectors in the top flange. The ultimate strength found in experimental tests was compared with calculation results according to BS 5400. The results showed a large margin between failure and buckling in the slender web girders of D/tw greater than 200. The ultimate shear ratio gap between test and calculated value ranged from 1.02 to 1.12. The ultimate loads of the steel girders were greater than the design loads by about 37%, while this was 29% to 35% for composite girders. Roberts and Al-Amery [19] investigated the effects of the shear connectors or bolted tension connectors on composite action. The investigation results revealed that adequate connection between concrete and steel enhanced the ultimate shear strength; failure occurred due to web buckling and plastic hinges in the top and bottom flange. The shear strength of the composite plate girder decreased linearly when the size of cut-outs was increased.

Baskar and Shanmugam [20] conducted an experimental investigation on steel-concrete composite plate girders under the combined action of moment and shear. Two parameters were studied, the slender web with values of Da/tw from 150 to 250, and the moment-shear ratio from 1.2 and 2.4. They observed that the failure due to shear load in the plate girders remained the same in the composite girders. The band of the tension field action was, however, wider in the composite girder than in the plate girders. Vasdravellis and Uy [21], studied the shear strength and shear-moment interaction of simply supported composite steel girders experimentally and numerically. The moment capacity in composite beams was reduced when the shear ratio, shear test value to the shear calculated value, reached 60%. Partial connections in the composite beams reduced the shear strength of the beams but increased the deformation in the ultimate strength. Yoo and Choo [22] proposed a composite girder of a slab of UHPC and a steel girder without a top flange. Their study arranged the studs in the web match the UHPC slab. The normal concrete slab developed some cracks and suffered a sudden failure when tested, while the thin slab member with larger spacing between studs suffered longitudinal
cracks due to redistributed shear stress around the studs. The slip between concrete and steel conformed to Eurocode4 and satisfied ductility requirements.

3. Experimental program

Plate girders and composite plate girders are commonly used to withstand flexural bending moment and shear forces. This study focused on the behaviour of the ultimate shear strength due to elastic buckling and diagonal tension action (post-buckling). Thus, the geometry of the specimens needed to be resistant to bending moment, with the moment to shear ratio being kept low to reduce the effects of flexural bending on the ultimate strength of the beam, as revealed by Roberts and Shahabian [16].

3.1. Specimen Geometry Details

3.1.1. Plate Girder. Seven plate girders were created by welding plate elements; the total length in each case was 2.2m, and the effective length was 2.1m. The flange thickness was 3.8mm for slender flange specimens, FW1, and 5.8mm for all other specimens, while web thickness was 2mm only for specimen FW2, being 1.2mm for all other web specimens. The variation in the state of the compression flange slenderness, \( \lambda_f \), was satisfied by changes in width; thus, the flange width was 225mm for FW5, 150 for FW3, and 75mm for FW4. The change of web slenderness, \( \lambda_w \), was satisfied by increasing the web depth and using a constant thickness for the web. The depth for FW3 was 450mm, for FW7 300mm, and for FW6 200mm. These different depths changed the web slenderness to 375, 250, and 167, respectively, as shown in Figure 2 and listed in Table 1. Specimens were fabricated from two flanges, and a web welded together with loading stiffeners in the mid-span only of similar thickness to the flange. Two pairs of bearing stiffeners were used at the ends of the beams. Transverse or longitudinal stiffeners were not used. The girders also had non-rigid post ends with free end distances equal to 50mm on each side, as shown in Figure 3.

![Figure 2. Cross Section Details of Flat Web Specimens.](image-url)
Table 1: Geometrical Properties of the Plate Girder Specimens.

| Designed | Top Flange width, \( b_T \) | Flange Thick. \( T_f \) | Bottom Flange width, \( b_b \) | Flange Thick. \( T_f \) | Web Depth \( D \) | Web thick. \( t_w \) | \( D/t_w \) | \( t/t_w \) | L/D | State of Flange |
|----------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|------------|------------|-----|----------------|
| FW1      | 200             | 3.8             | 200             | 3.8             | 420             | 1.2             | 350        | 3.33       | 5   | Slender       |
| FW2      | 175             | 5.8             | 175             | 5.8             | 450             | 2.0             | 225        | 3          | 4.67| Non-com.     |
| FW3      | 150             | 5.8             | 150             | 5.8             | 450             | 1.2             | 375        | 5          | 4.67| Non-com.     |
| FW4      | 75              | 5.8             | 225             | 5.8             | 450             | 1.2             | 375        | 5          | 4.67| Compacted    |
| FW5      | 225             | 5.8             | 75              | 5.8             | 450             | 1.2             | 375        | 5          | 4.67| Non-com.     |
| FW6      | 150             | 5.8             | 150             | 5.8             | 450             | 1.2             | 167        | 5          | 10.5| Non-com.     |
| FW7      | 150             | 5.8             | 150             | 5.8             | 300             | 1.2             | 250        | 5          | 7   | Non-com.     |

3.1.2. Composite Section. Three composite specimens were also created and tested. The main parameters studied included how the slenderness of the web affects ultimate shear strength in composite girders. Such variation in the slenderness was thus attained by increasing the depth of the web to 200, 300, and 450mm with constant web thickness, sequentially. This variation in the web depth varied the web slenderness, \( D/t_w \), to 167, 250, and 375, as shown in Figure 4. The composite sections consisted of a steel plate girder and UHPC deck slab, including steel fibre at 4.5% of friction volume to increase the ductility of the concrete. Each concrete deck slab was connected by two pairs of studs.

Figure 4: Cross Section Details of Composite girders.
3.1.3. Deck Concrete slab. The concrete slabs were formed from ultra-high performance steel fibre concrete with a fraction content of steel fibre of 4.5% and aspect ratio, length to diameter, of 65. The ingredients of the concrete were Portland cement type R42, fine aggregate passing through a 600-micrometer sieve size, and micro silica fume, these being mixed with superplasticizer and water before added steel fibre. After setting, the concrete specimens were cured by immersion in hot bath-water. These deck slabs were precast concrete, being prepared, and then connected to steel plate girders as explain in the three dimensions drawing in Figure 5 and the cross-section with longitudinal details shown in Figure 6. The proportions of the ingredients are listed in Table 2.

Table 2: Proportion of ingredients (per cubic meter).

|     | cement | Fine sand Kg | Rep. S.F. kg | Add. S.F. kg | w/cm | Super. kg | St. F.kg |
|-----|--------|--------------|--------------|--------------|------|-----------|----------|
|     | 900    | 1050         | 100          | 100          | 0.22 | 45        | 353.25   |

Figure 5: Three Dimension Composite Plate girder.

Figure 6: Typical Schematic of Composite Girder.

The width of the deck was 400mm, and the depth was 34mm, enlarged to 50mm in the middle of the slab to create a haunch height of 15mm, with a width of 150mm. A vibration table was used for good consolidation and levelling in the fresh concrete. After 36 hours, the concrete deck was de-moulded and cured using steam with the temperature increased gradually from 20°C to 60°C over three hours. The deck then underwent immersion in a hot bath-water for 28 days; the temperature of the water in the bath was about 60°C.
3.1.4. Shear Connector. The precast concrete slab was connected to the steel girder by a pair of studs to examine the composite behaviour between concrete and steel plate girders after setting and aging for 28 days. The connection was achieved by headed shear studs of 14mm diameter and 75mm depth. The studs were arranged as rows of two studs placed through the holes in the concrete and steel. The holes in the concrete had upper parts to a depth of 15mm with diameters of 30mm for the head of the stud while the remaining depth had diameter 15mm for the shank of the stud. The bottom parts of the holes in the concrete were prepared during casting, while the upper parts were drilled after concrete hardening. The holes in the top flange of steel plate girder were achieved just before connecting the concrete and steel, and the studs were tightened by use of a micrometer adjustable torque wrench. All nets were tightened at a torque moment of 140kN.m, and pairs of studs were distributed at 175mm intervals along the span, giving 12 pair of studs in each composite plate girder, as shown in Figure 6.

3.2. Measurement Devices

All specimens were tested using a compression machine, which was improved by adding an automatic control system. The LabVIEW 2018 program was used as a control system, and a set of Linear Variation Displacement Transducers (LVDT’s) were used in various positions to measure the variation in the distance during the testing process very precisely. The minimum division could thus be measured to 0.001mm without noise effects. Two LVDT’s were used for measuring the vertical deflection from different positions, while the other two sensors were used to measure the horizontal displacement in the middle of the top and bottom spans. During the process, the sensor was moved from the top middle span to the end of the girder to measure the relative slip displacement between concrete and steel during testing. Two types of strain gauge sensors were used, steel strain gauges and concrete strain gauges, respectively. These were produced by TML Tokyo Sokki Kenkyujo Co., Ltd [23]. Five strain gauges were used for the plate girders, and eight strain gauges were used for the composite plate girders. All instrument devices, including the load cell, LVDTs, and strain gauges, were connected by a single data acquisition (DAQ) type NI6052, itself controlled by one program, LabVIEW 2018. The data acquired during the test were saved automatically in Microsoft Excel and displayed on a monitor throughout testing.

3.3. Test procedures

The test setup is illustrated in Figure 7. The plate girders were subjected to one concentrated load at mid-span. The boundary conditions of the specimens were simply supported with lateral support conditions at the ends. The load applied in the mid-span of the specimens was measured by a load cell of 100ton capacity. Five uniaxial strain gauges were mounted on the surface of the steel according to manufacturing instructions, fixed at distances equal to a quarter of the web panel length (265mm) to avoid error due to direct stress from the applied load. Two uniaxial strain gauges were mounted, one of the top fibre surface of the steel flange and the other on the bottom fibre. The remaining three uniaxial strain gauges formed a strain rosette to measure the shear strain at the centroid of the section.

The number of strain gauges on the composite section was eight, with three uniaxial strain gauges mounted on the concrete deck slab. One of these was fixed to the bottom surface of UHPC at a distance of 70mm from the side edge, as shown in Figure 8, while the two strain gauges mounted on the top surface of UHPC were at the mid-width of the concrete and at a distance of 70mm from the side edge.
Figure 7: Test Setup: Steel Plate Girder.

Figure 8: Test Setup: Composite Girder.
4. Experimental Results

The mode failure of the steel plate girders was similar for all cases, as shown in Figure 12, with slight elastic buckling and then tension developing in the web panel, which governed the failure mode, except girders FW2, and FW6 underwent only to elastic buckling, the top flanges of the girders FW1, FW5 have buckled slightly in addition to web buckling since these flanges are slender. As the specimens had high flexural strength with low shear strength, the moment shear ratio was low, and the elastic buckling and then tension developing in the web panel, which governed the failure mode, except girders FW2, and FW6 underwent only to elastic buckling, as listed in Table 3. The top flanges of the girders FW1, FW5 have buckled slightly in addition to web buckling since these flanges are slender. As the specimens had high flexural strength with low shear strength, the moment shear ratio was low, and the shear strength was the controlling failure [21, 28, 29].

Table 3: The Results of Flat Web Girders.

| Designed | Elastic buckling load, E.B.L. kN | Deflection of Elastic Buckling, mm | Failure load, F.L. kN | Failure Deflection mm | E.B.L. to F.L. |
|----------|---------------------------------|-----------------------------------|----------------------|-----------------------|---------------|
| FW1      | 12                              | 2.8                               | 16.13                | 14                    | 16.7%         |
| FW2      | 57.7                            | 8.6                               | -                    | -                     | 0%            |
| FW3      | 27                              | 2.0                               | 42                   | 13                    | 55.6%         |
| FW4      | 39                              | 5.1                               | 40                   | 11.2                  | 2.6%          |
| FW5      | 14                              | 5.9                               | 22.24                | 17.5                  | 58.9%         |
| FW6      | 15.17                           | 4.3                               | -                    | -                     | 0%            |
| FW7      | 14                              | 1.9                               | 19.84                | 17.5                  | 41.7%         |

4.1. Load-Deflection Curve.

From the load-deflection curves, the loads and deflection varied in both magnitude and behaviour according to specimen properties. Testing revealed that the elastic buckling of the web panel occurred early, at between 100% to 61.1% of the total failure load. Therefore, the contribution of the post-buckling or tension field action in terms of resisting shear stress is minimal where webs do not have transverse stiffeners.

The FW1 specimen, which had a slender flange, showed local flange buckling near the support. This phenomenon was not seen in other specimens except FW5, which while it had a non-compacted flange, \( \lambda_p \leq \lambda_f \leq \lambda_r \) [1], had a slenderness ratio, \( \lambda_f \), equal to 8.75 when \( \lambda r \) was equal to 22.5, making its slenderness state close to that of the slender limit. For this reason, local buckling appeared in these specimens; all other remain specimens failed due to web buckling only. The load versus deflection curves were different in shape for each specimen, representing the different behaviours. The elastic buckling was most notable when the slope curve changed rapidly, as shown in Figure 10, Figure 9, and Figure 11.

Three states of slenderness were examined: the slender flange of FW1, the non-compacted flange of FW5, and the compacted flange of FW4. Figure 9 shows the schematic relationship between normalising shear and deflection. When the flange is in a compacted state, the girder gains shear strength, while when the flange is in a non-compacted flange state, the strength shear is lower. Girders with slender flanges have the lowest shear strength. When the depth to thickness ratio decreases, elastic buckling generally increases; however, the FW3 girder showed an aberrant result, possibly due to the interaction between flexural and shear strength, as shown in Figure 10. The percentage loads of the elastic buckling to ultimate load were 55.6% for web slenderness 375, 41.7% for web slenderness...
250, and 0% for web slenderness 167. Figure 11 showed the relations between normalize shear and deflection to depth ratio, which explained the effects of thickness ratio on the behaviour of the girders.

![Figure 9: Normalize Shear-deflection of the Girders FW3, FW4, and FW5.](image1)

![Figure 10: Normalize Shear-Deflection of the Girders FW3, FW6, and FW7.](image2)

![Figure 11: Normalize Shear-deflection of the FW3, FW1, and FW2](image3)

### 4.2 Composite Section Results

The composite girders were C.FW8, C.FW10, and C.FW9, with web depth to thickness ratios, D/tw, of 167, 250, and 350 respectively. The failure mode of all composite sections occurred on the web due to contribution effects of the elastic buckling and post-buckling, as shown in Figure 12. The elastic buckling of the girders C.FW9 and C.FW10 coincides, while the post-buckling behaviour of the C.FW8 coincides with composite girder C.FW10, as shown in Figure 13. In the composite sections, the failure load considered corresponds to deflection equaled to L/120, which is 17.5mm for all composite girders. When compared the percent increment of the post-buckling with elastic buckling, that percent reduced when the length to depth ratio increased, as listed in Table 4.

The load caused more elastic buckling of the composite section than of the plate girder because the deck slab reduces shear stress, which affects the web. Thus, the deck slab increased the resistance to
elastic buckling of the composite section. The post-buckling behaviour has a significant improved at composite section.

Figure 12: Failure Pattern of Flat Web Girders.

After elastic buckling, a hardening clearly shows due to post-buckling or tension field action. The percentages of elastic buckling loads were 26.6% for C.FW8, 20.5% for C.FW10, and 18.75% for C.FW9, with depth to thickness ratios of 167, 250, and 375, respectively, as listed in Table 4.

Figure 13: Normalize Shear Vs deflection of the Composite Section.

Figure 14: Mode of Failure for the Composite Girders.
Figure 15: Comparison between plate girders and composite girders.

Table 4: Elastic buckling and failure results of the composite girders.

| Designed | D/tw ratio | Elastic Buckling Load, E.B.L kN | Load corresponded L/120 deflection, 17.5mm, kN | Elastic buckling deflection, E.B.D., mm | E.B.L. to F.L. corresponded to L/120 |
|----------|------------|---------------------------------|-----------------------------------------------|---------------------------------------|---------------------------------------|
| C.FW8    | 167        | 15.8                            | 20                                            | 4                                     | 26.6%                                 |
| C.FW9    | 375        | 32                              | 38                                            | 4.5                                   | 18.75%                                |
| C.FW10   | 250        | 22                              | 26.5                                          | 4.5                                   | 20.5%                                 |

5. Conclusion

This study examined seven steel plate girders and three composite steel girders. All girders showed weak resistance to shear forces. The moment to the shear ratio is low, so the effects of flexural resistance are often underestimated. This study thus aimed to investigate the shear strength of plate girders and composite girders, and in particular to examine the effects of the slenderness of flanges (i.e., the neutral axis position) and the length to depth ratio on the shear strength capacity either elastic or post-buckling. The percentage of elastic buckling load to failure load effects were also discussed. The conclusions were as following:

1. The AISC equations for elastic buckling cover the shear strength of steel plate girders with a margin of safety.
2. All tested specimens failed in shear. The plate girder specimens failed due to elastic buckling with
very little tension field action, with the percentage of elastic buckling varying from 0% to 61.1%, for plate girders while the composite plate girders failed due to elastic buckling with much greater tension field action, and a load of elastic buckling varied from 18.75% to 26.6% of total failure load.

3. The shear strength of plate girder as compared with that of the composite girder had the same properties: the shear strength capacity of the composite section was greater than the shear strength of the plate girder due to post-buckling in the composite section. Thus, elastic buckling of the plate girders occurred earlier than in the composite sections due to the concrete slab, which reduces the shear stress in the web in the latter, as well as the changing in the properties of the cross-section such as the position of N.A. and length to depth ratio changed the behaviour of the plate girders and composite girders.

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