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Parametrical Study of Lateral Torsional Buckling Behaviour for Triangular Web Profile Steel Section

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Abstract: A triangular web profile (TRIWP) steel section is a structural steel section that is made up of two flanges connected to a web plate of triangular profile. As a newly proposed section, limited research have been carried out to study the characteristic, behaviour and the advantages offered by this section compared to the normal flat web (I-beam) and other types of corrugated web steel section. One of the area that still not been studied is the capability of this section to resist the lateral torsional buckling moment compared to the normal flat web. The objective of this thesis is to study the lateral torsional buckling behaviour of triangular web profile (TRIWP) steel section. This study involved modelling of 60 beam models (30 TRIWP and 30 FW) derived from four main model using LUSAS Modeller version 14.0 to investigate buckling resistance. There are three spans that have been used which are 3 m, 4 m and 4.8 m. For a conclusion TRIWP steel section has higher buckling moment resistance, Mcr compared to FW steel section.

Keywords: Finite element model, lateral torsional buckling, triangular web profile, steel

1. Introduction

The lateral torsional buckling (LTB) is a common mode of failure for structural steel section. Because of its importance in the design of beams and beam-columns, LTB continues to attract the attention of many researchers (Lopez et al. 2006; Prathebha and Jane Helena, 2018). A laterally unsupported compression flange will behave like a column and tend to buckle out of plane between points of lateral support. However, because the compression flange is part of a beam section with a tension zone that keeps the opposite flange in line, the section twists when it moves laterally. Simply said, it is a sidewise buckling of beam accompanied by twist.

One of the main issues in designing the steel structure is to reduce the weight and cost of the component parts. For economical purpose, thin webs are preferred in designing girder and beam. But the problems of plate buckling may arise if the web is extremely slender. To settle this problem, thicker plates can be used, web stiffeners can be added or using the latest innovative technique by strengthening the web by making it corrugated.

Bridges with corrugated web girders have been used since the late 1980's, especially in France and Japan. The use of corrugated steel plates as the web in bridge girders is an efficient way of achieving adequate out-of-plane stiffness and buckling resistance thus reducing the need for thicker plates and significantly reducing the number of stiffeners needed especially for transverse stiffeners that are needed during incremental launching of a bridge structure. Furthermore, high transverse stiffness of corrugated web allows for reducing the number of cross frames in box girder bridges. If the girder with sinusoidal web profile are used under working load condition, the section girder having the least moment reaction, deflection and stress value (Baby and Jacob, 2017).

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Nomenclature	
LTB	lateral torsional buckling
FCW	fully corrugated web
PCW	partial corrugated web
FW	flat web
TRIWP	triangular web profile

The trapezoidal corrugated web is designed for pure shear and does not contributed to the flexural capacity of the girder, which is undertaken by the top and bottom flanges including the concrete slab. For the two panels plate girders, an increment in the ultimate shear resistant of both composite plate girders are due to the increment of corrugation angle value. Studies have also been conducted on the effect of welding the cold formed corrugated web, which includes impact, fatigue and brittle facture testing. Atan (2001) had studied the flexural behaviour of trapezoidal web profile meanwhile Tan (2004) had done buckling analysis of compression member with trapezoid web profile. Osman et al. (2008) had studied the lateral torsional buckling of beam with trapezoid web.

There are several types of known corrugated web such as trapezoidal, sinusoidal and triangular. Trapezoidal and sinusoidal shape sections are quite popular in industry and among researchers compared to other corrugated web shapes. Figure 1 shows an example of sinusoidal web beam dimensions and designation while Figure 2 shows profiles of I-girder with trapezoidal webs. Shape and dimensions of a typical TRIWP section can be viewed in Figure 3.

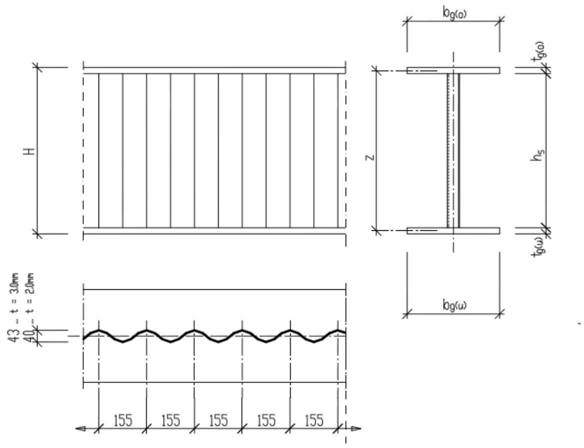


Fig. 1 - Example of sinusoidal web beam dimensions and designation (Zeman & Co 1999)

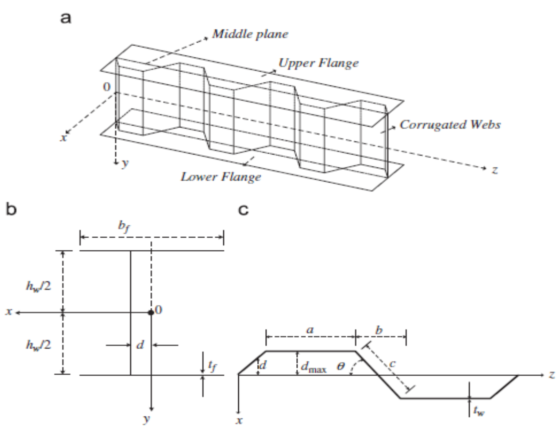


Fig. 2 - Profiles of I-girder with trapezoidal webs (Moon et. al, 2009)

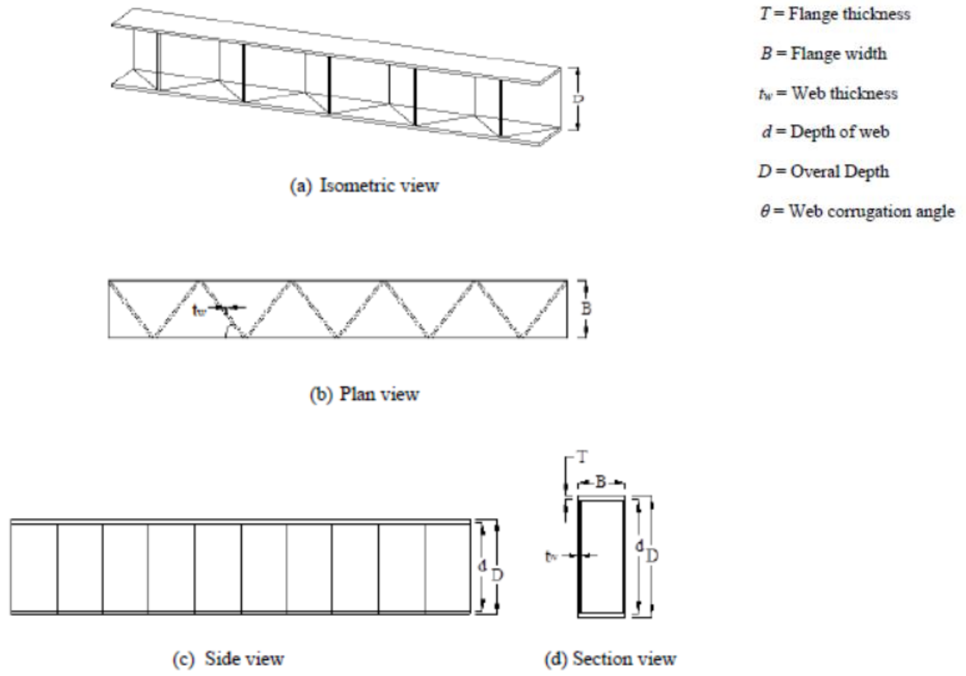


Fig. 3 - Shape and dimensions of a typical TRIWP section (all units are in mm) (Hashim et al. 2011)

Especially for the main frames of single-storey steel buildings, the use of corrugated web beams, mainly with sinusoidal corrugation, has been increased very much during the last years. Due to the thin web of 2 or 3 mm, corrugated web beams afford a significant weight reduction compared with hot rolled profiles or welded I-sections. Buckling failure of the web is prevented by the corrugation (Pasternak et al. 2010).

Triangular web profile steel section is a section consisting of two parallel flanges connected by triangular shaped web. Basically, it is just a modification of conventional flat web steel beam by replacing the flat web with triangular one. The depth of the section will determine the flange width and thickness of the section. For the section of a Partial corrugated web (PCW) and fully corrugated web (FGW), it is observed that significant increase of bending capacity of PCW and FCW steel beams when compared to flat web (FW) steel beam. Therefore, due to low fabrication cost consideration, it could be efficient to use PCW steel beams due to the practicable increment in the critical moment compared to FW steel beam and PCW steel beam (Abbas et. al, 2018).

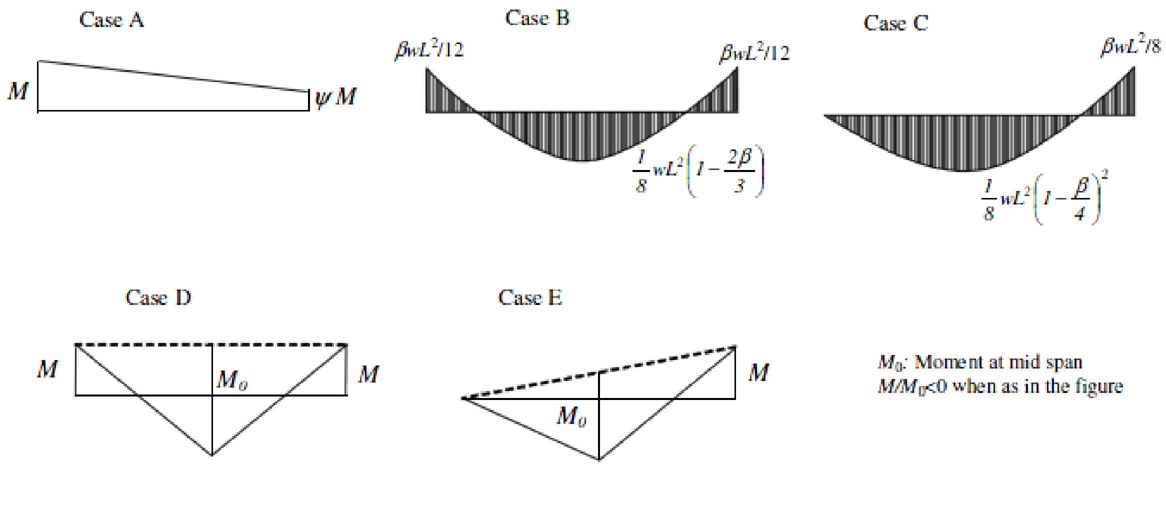


Fig. 4 - Moment diagram (EC3 2005)

The primary function of the top and bottom flanges plates is to resist the axial compressive and tensile forced caused by the applied bending moments. The main function of the web in a plate girder is to maintain the relative distance between the top and bottom flanges and resist the applied shearing force (Hassan 2011).

Various research works have been carried out to study the characteristic, behaviour and advantages offered by normal flat web section. Unfortunately, there is very small number of researches for the triangular web profile steel section. This is because triangular web is a newly proposed web section.

An area that is not given attention is the capability of triangular web profile to resist the lateral torsional buckling moment compared to flat web steel profile. Publication on lateral torsional buckling behaviour of triangular web profile steel section is none at the moment. The ability to resist lateral torsional buckling moment is one of the most important criteria to be considered when deciding to replace conventional flat web profile steel section. This ability will be one of main key indicator whether the triangular web profile steel section can be a good option or not.

Lateral torsional buckling of beams is considered by Eurocode 3 as an ultimate limit state related to member buckling resistance. The buckling resistance is obtained by multiplying the resistance of the cross-section by a reduction factor, χ_{LT}. This reduction factor is a function of two other parameters: the imperfections factor, α and the non-dimensional slenderness, $\lambda_{LT.}$

The parameter α takes into account the initial member imperfection, residual stresses and other nonlinear effects. The non-dimensional slenderness, λ_{LT} depends on the elastic critical moment for lateral torsional buckling, M_{cr}. However, Eurocode 3 does not provide information on how to compute M_{cr}. Moment diagram can be referred to Figure 4.

2. Methods

The method involves manual calculation according to Eurocode 3 and finite element analysis by using LUSAS software version 14.0. Manual design calculation was done to check the reliability of the modelling using LUSAS. LUSAS software use finite element method to analyse. The FW steel section was used as a control specimen because the web is flat and uniform in profile throughout the length. So, the second moment of area can be calculated easily and the manual calculation can be done. For TRIWP steel section, the second moment of area calculation procedure was not established yet. This study relies on the calculation made by LUSAS software using finite element method.

In this study, there were two models each for FW and TRIWP steel sections. The depth (D), breadth (B), web thickness (tw), flange thickness (tf) and corrugation angle (θ) are shown in the Table 1. The material coefficients to be adopted in calculation for the models were based on Eurocode EN 1993-1-1: 2005 clause 3.2.6. and they are shown in the Table 2.

2.1 Manual Design Calculation

Eurocode 3 applies to the design of buildings and civil engineering works in steel. It is based on limit state principles and comes in several parts. In this study, codes that were used are BS EN 1993 Part 1-1 and BS EN 1993 Part 1-5. To calculate design bending moment, fellow equations were used:

$$FEd = \gamma Q\ Qk \quad (1)$$
$$MEd = (FEd)pl \times span / span \div 2 \quad (2)$$

Where:

FEd = Design action
γQ = Partial safety factor for variable action
Qk = Variable action
MEd = Design bending moment
(FEd)pl = Design action for point load

Table 1 - The dimensional properties of models for analysis

Model	D (mm)	B (mm)	t_w (mm)	t_f (mm)	θ (°)
FW1	200	100	3	6	45
FW2	180	75	2	5	45
T_{RI}WP1	200	100	3	6	45
T_{RI}WP2	180	75	2	5	45

Table 2 - Material coefficient values

Coefficient	Value
modulus of elasticity, E	210 000 N/mm^2
shear modulus, G	81 000 N/mm^2
Poisson's ratio in elastic stage, v	0.3

For the section selection, refer to the bending moment value. Assume the section belongs to class 1. Plastic section modulus required about the major axis, $W_{pl,y}$ given by:

$$W_{pl,y} \geq M_{PL,Rd}\,\gamma_{M0} / f_y \quad (3)$$

$$\varepsilon = (235/f_y)^{0.5} \quad (4)$$

Where:

$M_{PL,Rd}$ = Design plastic resistance of cross section

γ_{M0} = Factor for resistance of cross section

ε = Strain

f_y = Yield strength

The standard equations for stress and strain for beams (flexure formulae) generally only consider the bending stresses and strains. The shear stresses are not considered. Class 1, plastic shear resistance, $V_{pl,Rd}$ is given by:

$$V_{pl,Rd} = A_v\,(fy/\sqrt{}\;\gamma_{M0}) \quad (5)$$

$$A_v = A\text{-}2bt_f + (t_w + 2r)\,t_f \geq \acute{\eta} h_w t_w \quad (6)$$

Where:

$\acute{\eta}$ = factor for shear area

h_w = depth, h-2t_f

t_f = thickness of flange

t_w = thickness of web

Due to high vertical stresses directly over a support or under a concentrated load, the beam web may actually crush or buckle as a result of these stresses. Elastic critical buckling, F_{cr} load given by:

$$F_{cr} = 0.9k_F E t^3_w / h_w \quad (7)$$

Where:

k_F = Buckling coefficient

E = Modulus of elasticity

To check deflection due to Q_k, following formula was used:

$$\delta = Q_k L^3 / 48EI \quad (8)$$

Where:

δ = Deflection

L = Length of beam

E = Young's Modulus

I = Moment of Inertia

2.2 Finite Element analysis

In this study, all models were assumed to buckle under perfect conditions, where there is no initial imperfectness and eccentric load. The buckling moments were then compared with result obtained from the manual calculation. Eigenvalue analysis of LUSAS Modeller was used to determine the buckling loads.

A linear buckling analysis is a useful technique that can be applied to relatively stiff structures to estimate the maximum load that can be supported prior to structural instability or collapse. The assumptions used in linear buckling analysis are that the linear stiffness matrix does not change prior to buckling and that the stress stiffness matrix is simply a multiple of its initial value.

The main objective in the eigenvalue buckling analysis is to obtain the critical buckling load, by solving the associated eigenvalue problem. The resulted eigenvalues are actually the load factors to be multiplied to the applied loading in order to obtain critical buckling load. The eigenvalue buckling analysis in LUSAS Modeller will provide both local and global buckling modes. Engineering judgment is necessary to determine which buckling mode is the most critical in order to select the appropriate buckling load factor. It is, of course possible to visually examine the resultant modes in LUSAS Modeller.

Convergence study had been done to get the optimum element size to mesh the model in order to get acceptable result without giving extra burden to the computer and lengthening the analysis process to an inappropriate level. The dimensional properties for the models used in this study can be referred in the Table 3 below. The results are displayed in Table 4 and Table 5. Typical model are shown in Figure 5 and Figure 6 detailed the support and loading condition.

Table 3 - The dimensional properties of models for the convergence study

Model	D (mm)	B (mm)	t_w (mm)	t_f (mm)	θ (°)	Span (mm)
T_{RI}WP1	200	100	3	6	45	600
T_{RI}WP2	180	75	2	5	45	600

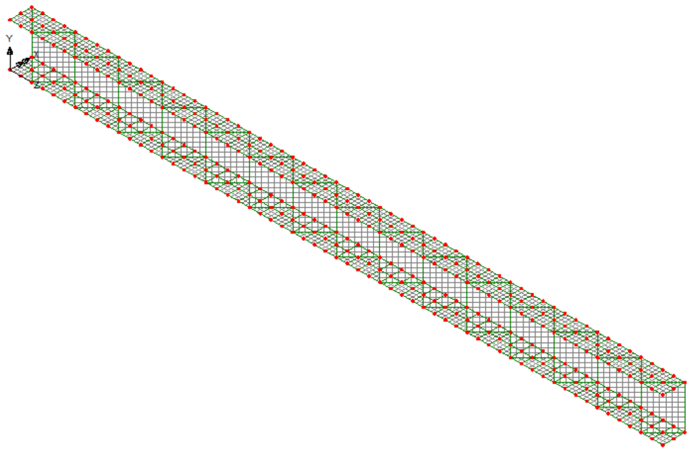


Fig. 5 - Example of model after assigning attributes

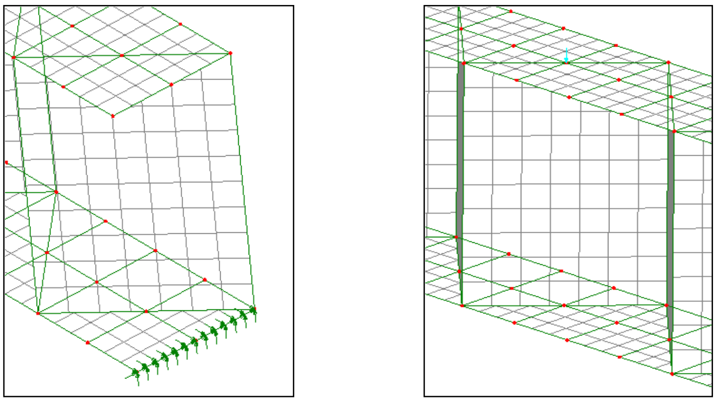


Fig. 6 - Mesh assignment (a) support condition; (b) loading condition

Table 4 - Data of maximum nodal displacements of T_{RI}WP1 model

Model	Size of elements	Number of elements	Number of nodes	Displacement (mm)
1	10	2880	9117	48.71
2	20	852	2817	48.73
3	30	402	1381	48.79
4	40	168	607	49.10
5	50	120	449	49.10
6	60	84	327	49.10

Table 5 - Data of maximum nodal displacements of T_{RI}WP2 model

Model	Size of elements	Number of elements	Number of nodes	Displacement (mm)
1	10	1956	6249	55.51
2	20	462	1565	55.60
3	30	192	681	55.97
4	40	138	505	55.96
5	50	96	365	55.96
6	60	84	327	55.95

The results show that a convergence solution has been obtained when the number of elements is 852 for TRIWP1 and 340 for TRIWP2. So the element size 20 is small enough to get a good result. This value was used to mesh all the models for numerical analysis.

Then, the analysis of 60 beams was run by the following steps:

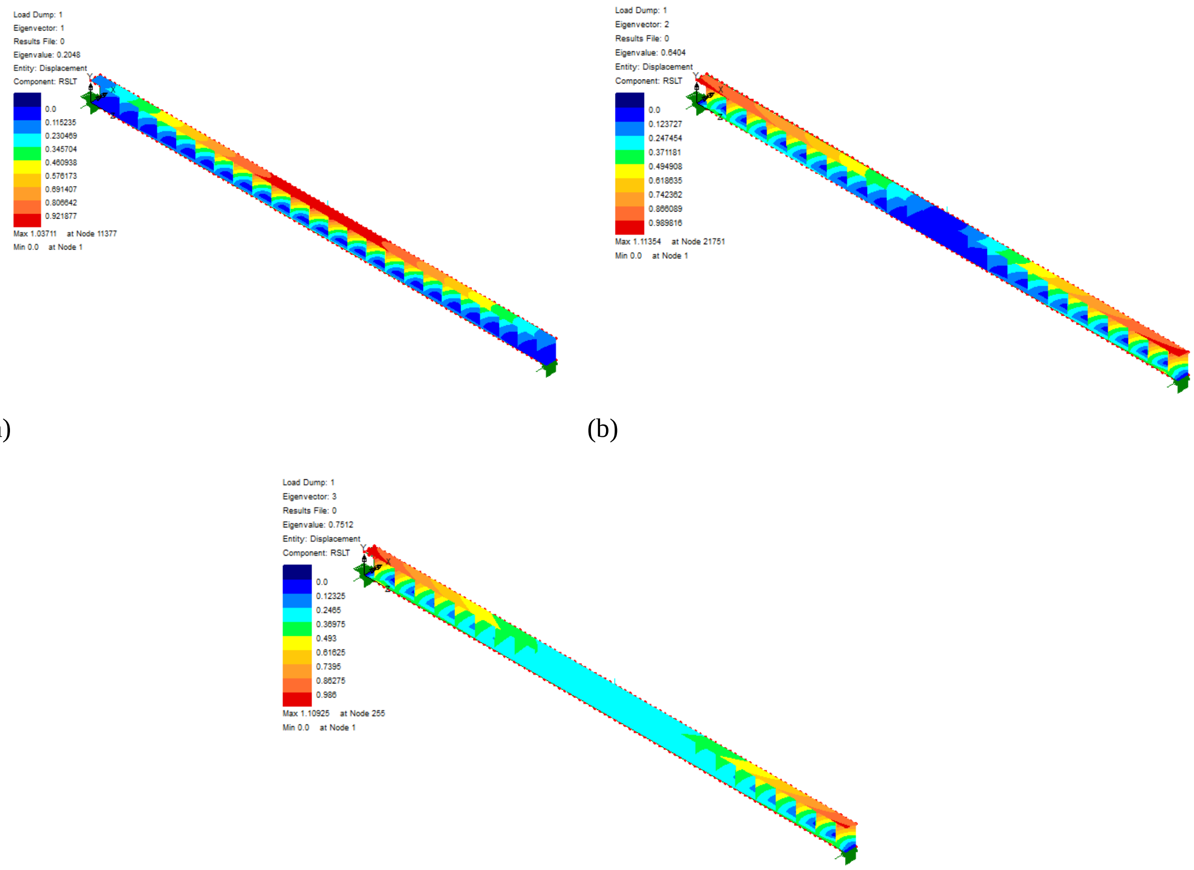
1) New model was created.
2) The geometry of the model was drawn. In this study, all the TRIWP steel sections modelled were applied corrugation angle value of 45^{o}.
3) Attributes were set up. The structural element descriptions are thin shell structural element, quadrilateral element shape and quadratic interpolation order. Regular mesh has been ticked and the element size was set up as 20. The value of the element size was decided based on the convergence study that has been done before. In this study, the value of P was 50 kN for all the models. The value was signed negative to make the direction downward in y-axis.
4) All the attributes were dragged to the section. The structural support was assigned at the bottom end of tension flange of the beam. The support was equally applied to both ends. Meanwhile, the loading was assigned at the inclined part of the corrugation at the mid-span.
5) Eigenvalue was set up. The solution type chosen was buckling load and the number of eigenvalues applied was three. Eigenvalues required was set as minimum.
6) The model was saved and the analysis was run.

3. Results

The numerical analysis originally consists of four main models of FW1, FW2, TRIWP1 and TRIWP2. In order to study the effect of web thickness to the buckling behaviour of the main models, four more models were derived from each of them by varying the web thickness of the steel sections. Furthermore, there will be three spans used which are 3 m, 4 m and 4.8 m of length which subsequently tripled the number of models. For overall, there are 60 beams analysed numerically using LUSAS software. By having more varieties in span and web thickness, the dependability of the result can be checked by using engineering judgement.

This linear buckling analysis can be applied to relatively stiff structures to estimate the maximum load that can be supported prior to structural instability or collapse. The assumptions used in linear buckling analysis that the linear stiffness matrix does not change prior to buckling and the stress stiffness matrix is simply a multiple of its value (De'nan 2008). Typical result is shown in Figure 7(a), 7(b) and 7(c). Figure 7(a), 7(b) and 7(c) show the results for analysis of beam model TRIWP1 with span 4.8 m.

From three eigenvalues of each model, the one from the first mode is chosen. This is because, the lower the value of the eigenvalue the more probability of the mode failure to occur. The eigenvalues chosen then will be time with the applied load, P to obtain their buckling loads, P_b. A sample of detail calculation of P_b can be referred below.



(a) (b) (c)

Fig. 7- (a) Result for eigenvalue 1; (b) result for eigenvalue 2; (c) Result for eigenvalue 3

3.1 Comparison of Buckling Load

The results obtained from the buckling analysis are plotted in the graphs to give better views and to make the comparison easier. Graphs of buckling load versus web thickness of each span are plotted as can be seen in Figure 8 to 13. Each graph consists of results from analysis of ten beams with the same length of span having two different web profiles (TRIWP and FW) and five web thickness sizes.

The web thickness sizes ranging from one to five mm for FW1 and TRIWP1 models, meanwhile the web thickness sizes for model FW2 and TRIWP2 are ranging from one to three mm.

Based on the Figure 8, 9, 10, 11, 12, and 13, it is obvious that the TRIWP steel sections have higher buckling load, Pb compared to FW steel sections at most of the cases. But there are some cases that the FW have higher value of Pb. As referred to Figure 9, 11 and 13 the FW have higher value of Pb compared to TRIWP when the web thickness is 1 mm. this may be caused by the geometry of the TRIWP. TRIWP steel section is a non-uniform section. So, the non-uniformity of the web section may induce instability and contributing to the buckling of the section at the support.

For overall, we can conclude that TRIWP has higher buckling load, Pb compared to FW. The results indicate that the slanting web of TRIWP steel section gives significant contribution to its stiffness.

Other than that, Figure 8 to 13 also show that the buckling load, Pb increased when the web thickness is increased. The result seems to be correct because theoretically the increase in web thickness will increase the stiffness of the section.

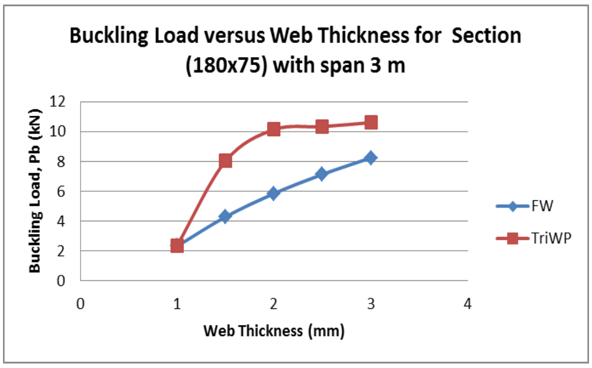


Fig. 8 - Graph of buckling load versus web thickness for section (180x75) with span 3 m

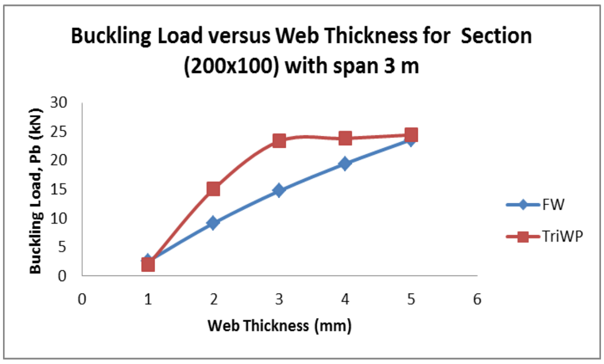


Fig. 9 - Graph of buckling load versus web thickness for section (200x100) with span 3 m

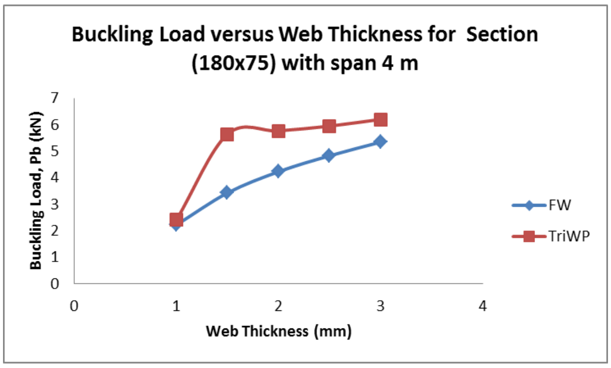


Fig. 10 - Graph of buckling load versus web thickness for section (180x75) with span 4 m

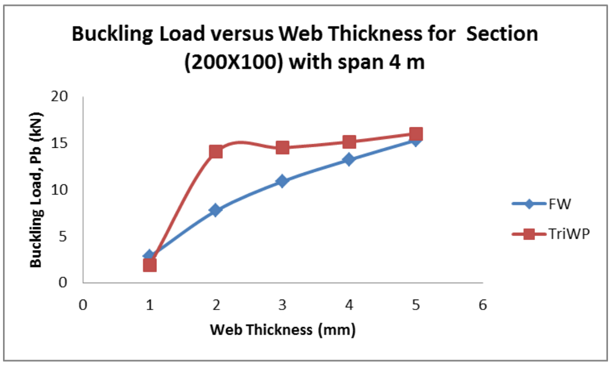


Fig. 11 - Graph of buckling load versus web thickness for section (200x100) with span 4 m

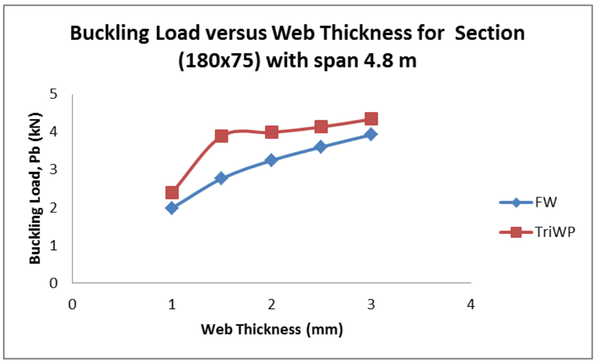


Fig. 12 - Graph of buckling load versus web thickness for section (180x75) with span 4.8 m

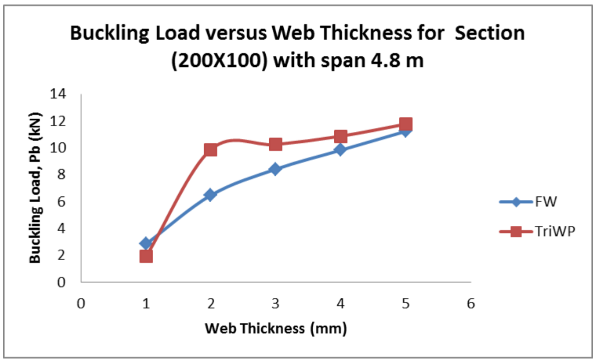


Fig. 13 - Graph of buckling load versus web thickness for section (200x100) with span 4.8 m

3.2 Comparison of Buckling Moment

From the buckling loads, critical buckling moments, M_{cr} are calculated by timing the values with length of span and dividing them with four. As an example, for TRIWP1 with span 3 m, M_{cr} = 23.34 x 3/4. The results are shown in Table 6.

Percentage difference is calculated by subtracting the value of M_{cr} for TRIWP with the value of M_{cr} for FW, dividing the value of the subtraction with the value of Mcr for FW and then timing with 100. As an example, for FW1 and TRIWP1 with span 3 m, percentage difference = 100(17.50 - 11.04)/11.04.

Based on the results from the Table 6, it is proven that the TRIWP has higher critical buckling moment value compared to FW. Both TRIWP models show higher value of M_{cr} compared to FW models with the same section size regardless of the span length.

Result from Table 6 also shows that longer span gives lower value of M_{cr}. This is because the distance between concentrated load at mid-span and the support increase as the length of span increase.

It is also noted that the percentage difference of M_{cr} between FW and TRIWP decrease with the increase of length of span. This result can be used as an indicator whether TRIWP can substitute the FW in the industry because one of the major reasons of using corrugated web beam is to allow the use of lengthier span in structure. Study can be done in the future to investigate the effect of length of span towards the percentage difference of M_{cr} between FW and TRIWP using lengthier spans

Table 6 - Values of critical buckling moment, M_{cr} for four main models and percentage difference

Span (m)	Model	M_{cr} (kNm)	Percentage difference (%)
3	FW1	11.04	58.51
	T_{RI}WP1	17.50	
	FW2	4.39	73.08
	T_{RI}WP2	7.60	
4	FW1	10.89	33.11
	T_{RI}WP1	14.49	
	FW2	4.23	36.42
	T_{RI}WP2	5.76	
4.8	FW1	10.09	21.80
	T_{RI}WP1	12.29	
	FW2	3.89	22.82
	T_{RI}WP2	4.78	

4. Conclusion

Based on the overall result, judgements and discussions, the following conclusions have been made from this study:

1. Numerical analysis using LUSAS software has been used successfully to estimate the critical buckling load of lateral torsional buckling of the member. It can be concluded that buckling moment resistance, Mcr of the triangular web (TRIWP) profile steel section is higher compared to flat web (FW) profile steel section

2. The behaviour of the triangular web (TRIWP) profile steel section had been studied using finite element software and the section had been modelled successfully. The LUSAS software has proven to be a reliable tool to analyse structural member including the newly proposed triangular web (TRIWP) profile steel section. This will help in order to understand the behaviour of the new section before it can be commercialised.

3. The effect of web thickness and length of span towards buckling moment resistance of triangular web profile steel section had been studied. The results indicate that the increase in web thickness will improve the buckling moment resistance of the beam; meanwhile shorter span will produce higher buckling moment resistance value.

As an overall conclusion, this research work has fulfilled the objectives. But there might be some weaknesses in this study that can be cater by future researchers. The time constrains and budget limitation has reduced the scope of the study.

There are parameters that have not been covered in this study which are the effect of corrugation angle and the depth of beam. The high cost of steel in the market had been a big obstacle to improve the research work. This is because, experimental work using real steel structure is important to be performed in order to validate the numerical analysis result.

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