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Effects of plate slenderness on the ultimate strength behaviour of foam supported steel plate elements

Narayan Pokharel† and Mahen Mahendran‡

School of Urban Development, Queensland University of Technology, Brisbane, Australia

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Abstract. Plate elements in fully profiled sandwich panels are generally subjected to local buckling failure modes and this behaviour is treated in design by using the conventional effective width method for plates with a width to thickness (b/t) ratio less than 100. If the plate elements are very slender ($b/t >$ 1000), the panel failure is governed by wrinkling instead of local buckling and the strength is determined by the flexural wrinkling formula. The plate elements in fully profiled sandwich panels do not fail by wrinkling as their b/t ratio is generally in the range of 100 to 600. For this plate slenderness region, it was found that the current effective width formula overestimates the strength of the fully profiled sandwich panels whereas the wrinkling formula underestimates it. Hence a new effective width design equation has been developed for practical plate slenderness values. However, no guidelines exist to identify the plate slenderness (b/t) limits defining the local buckling, wrinkling and the intermediate regions so that appropriate design rules can be used based on plate slenderness ratios. A research study was therefore conducted using experimental and numerical studies to investigate the effect of plate slenderness ratio on the ultimate strength behaviour of foam supported steel plate elements. This paper presents the details of the study and the results.

Key words: profiled sandwich panels; local buckling; flexural wrinkling; plate slenderness limit; intermediate region; effective width.

1. Introduction

With the rapid advancements in manufacturing technology, thin and high strength steel plates are being used in many structural and building systems. As they are very economical and exhibit very high strength to weight ratio, their popularity has increased considerably among the designers and manufacturers of Australian construction industry. Therefore thin and high strength steel plates are increasingly used in sandwich panel construction. Due to the use of such thin plates, the plate elements in sandwich panels are becoming more slender. Generally the plate elements in the fully profiled sandwich panels are subjected to local buckling effects under compression or bending actions (see Fig. 1). In the current design practice, the local buckling phenomenon of the fully profiled sandwich panels is treated by using the well known effective width method, originally developed by Winter (1947) for the plain plate elements without the foam core. Davies and Hakmi

† PhD Student
‡ Professor, Corresponding author, E-mail: m.mahendran@qut.edu.au

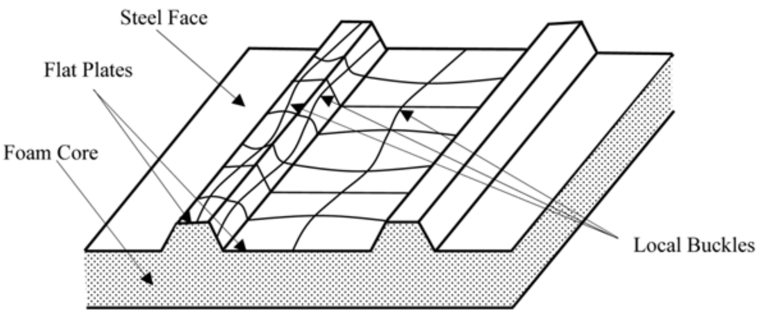

Fig. 1 Local buckling of profiled sandwich panels

(1990, 1992) conducted a series of bending tests on the foam-filled steel beams in order to investigate the concept of extending the effective width approach to foam supported steel plate elements. Based on their investigation, a modified effective width method was developed for sandwich panels subjected to local buckling effects based on a modified buckling coefficient *K*, which depends on the properties of the steel face and core material. After taking into account the non-linear behaviour of foam properties and other material uncertainties, this method was further modified and recommended as an acceptable design method for the local buckling behaviour of sandwich panels (CIB 2000).

Recently, Pokharel and Mahendran (2003, 2004a and 2004b) investigated the local buckling behaviour of fully profiled sandwich panels using a series of experiments and finite element analyses. This study showed that the current effective width formula (CIB 2000) can be successfully applied to the foam-supported steel plate elements with low width to thickness (*b/t*) ratios, generally less than 100. However, it can not be extended to slender plate elements, which are very commonly used in the Australian sandwich panel construction because of the use of very thin high strength steel plates. Therefore, in order to eliminate this inadequacy associated with the current design method, improved design formulae were developed for the safe design of fully profiled sandwich panels (Pokharel and Mahendran 2004a, 2004b).

In order to use the appropriate design rule for the foam supported thin plate elements, the buckling and ultimate strength behaviour of steel plate elements with increasing *b/t* ratios must be fully understood. As mentioned earlier, if the *b/t* ratio of the plate element is comparatively low, plate elements are subjected to local buckling modes. Local buckling does not mean member failure. There is considerable postbuckling strength beyond elastic local buckling which is taken into account in design using conventional effective width principles. However, if the *b/t* ratio of the plate element in sandwich panels is very high, its strength will be governed by wrinkling failure and can be determined by the well established flexural wrinkling stress formula (CIB 2000). Flexural wrinkling of sandwich panels is a form of local instability of compression steel faces associated with short waves of buckling. Unlike local buckling, flexural wrinkling failure does not include any postbuckling strength, and occurs in the elastic region at a stress well below the yield stress of steel. The strength of the slender plates, the *b/t* ratio of which lies in this wrinkling region, is very low compared with the local buckling strength. The strength of compact plates whose *b/t* ratio lies in the local buckling region is dominated by considerable postbuckling strength. However, if the *b/t* ratio of the plate element lies in the intermediate region (i.e., between the local buckling and wrinkling regions), the wrinkling formula can not be used as it underestimates the true strength of the plate.

Current effective width formula (CIB 2000), which is based on the postbuckling strength of plate element in the local buckling region, overestimates the strength of the panels in the intermediate region as indicated by Pokharel and Mahendran (2003).

Plate elements in profiled sandwich panels generally used in many sandwich constructions lie in this intermediate region. Plate elements in this region exhibit either very low or no postbuckling strength in the earlier part of the region or fail by wrinkling in the latter part of the region. Until recently, no guidelines were available to define the appropriate plate slenderness limit in order to separate the local buckling and wrinkling regions. Therefore it is necessary to investigate and identify the appropriate plate slenderness limit so that suitable design formulae can be used to obtain safe design solutions. To achieve this, a detailed investigation of the structural behaviour of foam-supported steel plate elements with low (local buckling region) to very high (wrinkling region) plate slenderness ratio (b/t) is required. In this study, the buckling and ultimate strength behaviour of polystyrene foam supported high strength steel plate elements with increasing b/t ratios up to 2000 was investigated using a series of experiments and finite element analyses and this paper presents the details of the study and the results.

2. Design methods for fully profiled sandwich panels

This section presents a brief overview of the important strength equations used for the design of plate elements in fully profiled sandwich panels with increasing plate slenderness ratio (b/t). Local buckling of the plate elements is the common failure mode of fully profiled sandwich panels. In the effective width method of design, the ultimate strength of the plate elements in sandwich panels subjected to axial compression, bending or combination of theses two actions, is determined based on the effective width of the plate elements in compression, which takes into account the postbuckling strength. The width b of the compressed plate element is replaced by a reduced effective width b_{eff} when calculating the section properties in the design calculations. So the determination of effective width of the compressed plate element in fully profiled sandwich panels is the most important step in determining the compressive strength of the panel (Davies 2001). The effective width formula takes the following form:

$$\frac{b_{eff}}{b} = \frac{1}{\lambda}\left[1 - \frac{0.22}{\lambda}\right] \tag{1}$$

$$\lambda = 1.052\left[\frac{b}{t}\right]\sqrt{\frac{f_y}{E_f K}} \tag{2}$$

where b_{eff} is the effective width of plane parts of a face profile, b the actual width, t the thickness of the steel plate, f_y the yield stress of steel, E_f the modulus of elasticity of steel and K the buckling coefficient of foam supported steel plates. This effective width formula is similar to that used for the plain plate elements except the buckling coefficient K, which is determined based on the plate slenderness ratio (b/t) and the material properties of foam and steel (E_c, G_c, E_f, v_f). Davies and Hakmi (1990) extended this approach to the sandwich panels based on their experimental investigation on foam-filled steel beams. They developed the formula to determine the modified value of buckling coefficient K and further improved it for design purposes. In the current European design method (CIB 2000), the following explicit formulae for K, proposed by Davies and Hakmi (1990), has been recommended to calculate the design values of effective width using Eqs. (1) and (2).

$$K = [16 + 7R + 0.02R^2]^{1/2} \tag{3}$$

$$R = \frac{12(1 - \nu_f^2)}{\pi^3}\frac{\sqrt{E_c G_c}}{E_f}\left[\frac{b}{t}\right]^3 \tag{4}$$

where E_c and G_c are Young's modulus and shear modulus of foam core, respectively, E_f the modulus of elasticity of steel and ν_f the Poisson's ratio of steel. Pokharel and Mahendran (2003) found that the effective width method described in Eqs. (1) and (2) using the buckling coefficient K defined by Eq. (3) is applicable only for the plates with a low b/t ratio (b/t < 100) and it can not be extended to slender plates. However, the effective width method could be extended to the plate elements with a high b/t ratio if a more appropriate formula is developed for the evaluation of K values. Pokharel and Mahendran (2003) proposed the following formula for K in order to calculate the effective width of plate elements including slender plates using Eqs. (1) and (2).

$$K = [16 + 1.18R + 0.00055R^2]^{1/2} \tag{5}$$

To further improve the design method, Pokharel and Mahendran (2004a) conducted a detailed finite element study on foam supported steel plate elements subject to local buckling effects. Based on this investigation, a new design method was developed as given next for plate elements with any practical plate slenderness.

$$\frac{b_{eff}}{b} = \frac{0.34}{\lambda}\left[1 + \frac{7.71}{\beta} - \frac{12.72}{\beta^2} + \frac{5.35}{\beta^3}\right] \tag{6}$$

$$\beta = \lambda\sqrt{K} \tag{7}$$

$$K = [16 + 11.8R + 0.055R^2]^{1/2} \tag{8}$$

where λ and R are as defined in Eqs. (2) and (4), respectively.

If the b/t ratio of the foam supported plate element in fully profiled sandwich panels is very large, then the plate element will fail due to flexural wrinkling instead of local buckling. The strength of such plate elements should be determined by the wrinkling formula as the above mentioned effective width formula will overestimate their strength. A well established theoretical wrinkling stress formula developed for the flat faced sandwich panels based on the elastic half-space method can be given by:

$$\sigma_{wr} = 1.89\left\{\frac{8(1 - \nu_c)^2}{12(1 - \nu_f^2)(1 + \nu_c)(3 - 4\nu_c)^2}\right\}^{1/3}(E_f E_c G_c)^{1/3} \tag{9}$$

Because of the practical considerations such as imperfections, material non-linearity, and inadequacy of the analysis, the theoretical wrinkling stress is not achieved and wrinkling failure takes place at a stress lower than that predicted by Eq. (9). Due to these constraints, the wrinkling stress σ_{wr} is calculated using Eq. (10) for practical purposes.

$$\sigma_{wr} = k_{wr}(E_c G_c B_f)^{1/3} \tag{10}$$

where k_{wr} is a numerical constant and may be determined experimentally for a particular product.

3. Compression tests of foam supported steel plates

To identify the plate slenderness limit that separates local buckling, wrinkling and intermediate regions, it is necessary to fully understand the buckling and ultimate strength behaviour of foam-supported steel plate elements with a *b/t* ratio ranging from very small to high values. Hence, a series of experiments on foam-supported steel plates with *b/t* ratio ranging from 200 to 1000 was conducted to investigate their behaviour. Test specimens were made by gluing a steel plate to foam core. Details of the test specimens used and the experimental program are given in Table 1 and Fig. 2.

Steel plates of all the test specimens were made of G550 grade. In the evaluation of experimental results, the experimentally measured mechanical properties of foam core and steel plates (Pokharel and Mahendran 2003) were used in this investigation. These experimental values for foam core were E_c = 3.8 MPa, G_c = 1.76 MPa, and ν_c = 0.08. Similarly, the experimental values of Young's modulus and yield stress for G550 grade steel with different thicknesses were taken from Table 1. The Poisson's ratio of steel was taken as 0.3. From the study conducted by Mahendran and Jeevaharan (1999) and Mahendran and McAndrew (2003), it was discovered that the thickness of foam core had negligible effect on the strength results. Therefore a constant foam thickness of

Table 1 Test program and specimens used in this study

Test no.	Plate width b (mm)	Base metal thickness t (mm)	Measured		b/t ratio
			f_y (MPa)	E_f (GPa)	
1	120	0.60	682	235	200.0
2	150	0.60	682	235	250.0
3	180	0.60	682	235	300.0
4	200	0.60	682	235	333.3
5	150	0.42	726	239	357.1
6	180	0.42	726	239	428.6
7	200	0.42	726	239	476.2
8	300	0.60	682	235	500.0
9	260	0.42	726	239	619.1
10	300	0.42	726	239	714.3
11	340	0.42	726	239	809.5
12	380	0.42	726	239	904.8
13	420	0.42	726	239	1000.0

Foam Properties: E_c = 3.8 MPa, G_c = 1.76 MPa, ν_c = 0.08

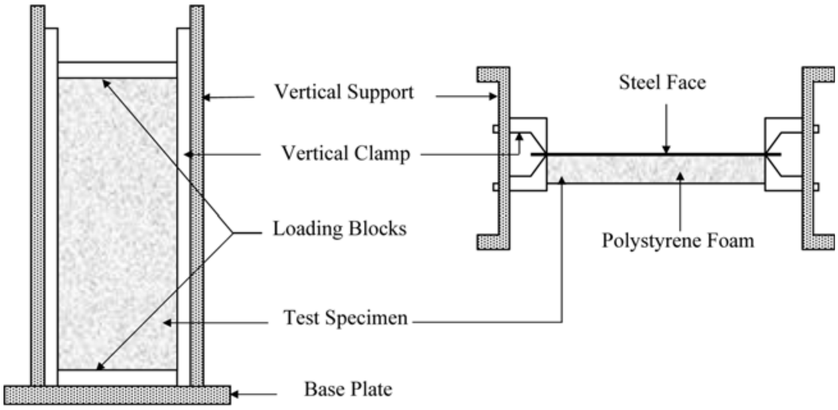

Fig. 2 Schematic diagram of test rig

100 mm was chosen in all the experiments irrespective of the plate widths used. To obtain high *b/t* ratios, a lower thickness of 0.42 mm was used in most of the specimens.

The plate elements in the profiled faces of sandwich panels can be represented as a simply supported plate subjected to an applied pressure p along the two transverse edges. In other words, their longitudinal edges are assumed to be simply supported. In order to model the required simply supported boundary conditions along the longitudinal edges, a specially constructed test rig was used to hold the test specimen during the compression test. Furthermore, these simply supported boundary conditions along the longitudinal edges of the plate were designed to simulate the real conditions present on the plate elements of the profile faces supported by adjoining plates. In order to use a wide range of plate widths and lengths, two different sizes (small and large) of test rigs were constructed. The small test rig was used to hold small specimens with the widths and lengths less than 200 mm and 600 mm, respectively, whereas the large test rig was used for the specimens with widths and lengths up to 420 mm and 1300 mm, respectively. Both the test rigs consisted of a base plate and two vertical supports. The vertical supports were adjustable both in horizontal and vertical directions to accommodate the required plate widths and lengths. The vertical clamps allowed shortening of the plates and rotation about the vertical edges to occur freely, hence adequately representing the simply supported conditions of longitudinal edges. Fig. 2 shows the schematic diagram of the test rig used in the experiments. Because of the width and height limitations with the available testing machine, plate elements with *b/t* ratio more than 1000 could not be tested. Test specimens were held in the test rig between the two loading blocks. The length of the loading blocks was made equal to the width of the test specimens. It must be noted that all the foam-supported specimens used in this investigation had only one steel face. The load was applied directly to this single steel face eliminating any possibility of uneven load distribution that might occur in the specimens with two steel faces. If two steel faces are used, the compression load will be distributed equally to both faces and the ultimate load of each face will be about the same whether the panel has one or two faces. Hence it is considered that the results are not influenced by the use of one steel face as used in this study.

To undertake the compression test on foam-supported steel plate elements, the test rig holding the test specimen between the two loading blocks was placed in the Testing Machine. The axial compression load was then applied to the steel plate via the top loading block. Arrangements were

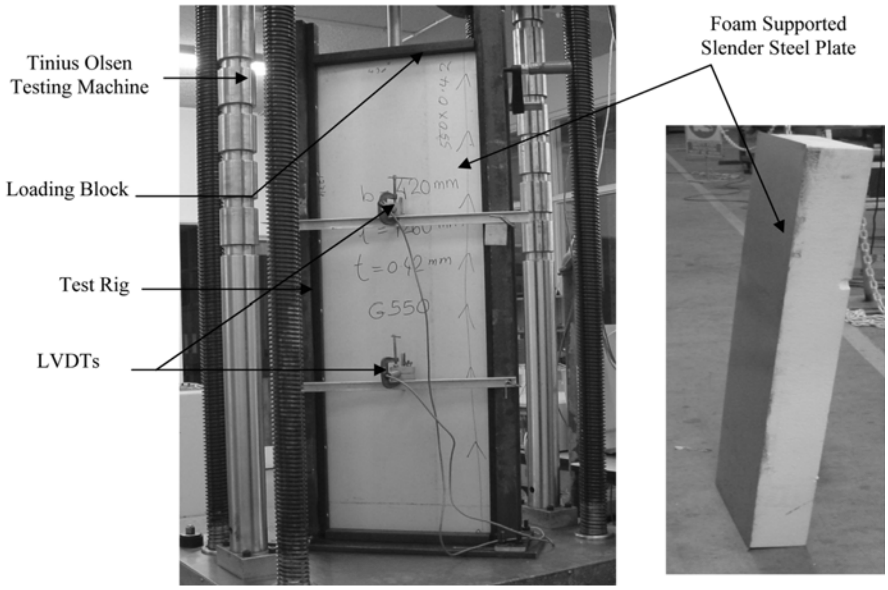

Fig. 3 Test set-up for foam-supported slender plate specimens

Table 2 Comparison of ultimate stresses from experiments and half-length models

Test no.	Plate thickness t (mm)	b/t ratio	Ultimate stress (MPa)	
			Experiment	FEA
1	0.60	200.0	169.86	152.22
2	0.60	250.0	133.89	133.22
3	0.60	300.0	122.59	124.35
4	0.60	333.3	118.00	118.50
5	0.42	357.1	119.21	133.33
6	0.42	428.6	118.39	124.87
7	0.42	476.2	100.12	120.60
8	0.60	500.0	87.44	99.00
9	0.42	619.0	99.82	106.59
10	0.42	714.3	96.27	100.00
11	0.42	809.5	94.54	94.26
12	0.42	904.8	79.32	90.48
13	0.42	1000.0	51.02	87.13

made to measure the axial compression load, axial shortening and out-of-plane deflection. The complete arrangement of the test set-up is shown in Fig. 3. The compression load was applied at a rate of 0.5 mm/minute until the failure of the specimen occurred. It is to be noted that the compression load was applied to the single steel plate element only and not to the foam core. The ultimate load which was the maximum load carried by the specimen was recorded by the testing machine. The ultimate strength results for all the specimens obtained from the tests are given in Table 2 where they are compared with the corresponding finite element analysis results.

4. Finite element analysis

4.1 FEA model

All the foam supported slender plate elements tested in this experimental program were also investigated using an extensive series of finite element analyses. In this study, the finite element program ABAQUS (HKS 1998) was used to model and analyse sandwich panels subjected to local buckling effects because of its extensive capabilities and availability. MSC/PATRAN was used for pre-processing (model generation) and post-processing (visualisation of results) phases of modelling. Since the thin steel plate elements used in the sandwich panels are subjected to buckling effects, the chosen element must be capable of modelling buckling deformations and associated behaviour. For the plate element with a low b/t ratio, the ultimate capacity is governed by local buckling and postbuckling behaviour. The postbuckling phenomenon is a complicated behaviour and occurs beyond the elastic region. So the element must be capable of modelling structural behaviour both in linear and non-linear regions involving large displacements, elasto-plastic deformations and associated plasticity effects. In the ABAQUS element library (HKS 1998), the shell elements generally satisfy these criteria and can be used to model the steel plate elements of profiled sandwich panels. Hence, the steel plate element was modelled using S4R5 three dimensional thin shell elements with four nodes and five degrees of freedom per node. It is a small-strain thin shell element and can model large deflections and rotations accurately. Thin shell elements are normally used in cases where transverse shear flexibility is negligible and the thickness of the shell is less than about 1/15 of a characteristic length on the surface of the shell, such as the distance between the supports or the wave length of a significant eigenmode (HKS 1998). Steel faces used in the sandwich panels fall well within this category and hence, 3D thin shell elements S4R5 with reduced integration were used successfully to model the steel plate elements. The foam core was modelled using C3D8 three dimensional solid (continuum) elements with eight nodes and three degrees of freedom per node. These elements, which have no rotational degrees of freedom, are also called 8-node linear bricks. Since there was no relative movement between the steel face and foam core, they were modelled as a single unit. For plates with a high b/t ratio, wrinkling instability occurs and is followed by degeneration into a wrinkle associated with yielding. In this case also the 3D thin shell elements S4R5 were used successfully.

The methods of analysis used for the investigation of local buckling behaviour of the foam-supported steel plate elements were elastic buckling analysis and non-linear analysis. Elastic buckling analysis is a linear perturbation analysis used to obtain eigenvalue-buckling estimates. The critical buckling stress, buckling shape and half-wave buckle length required to create the half-wave buckle length model were obtained from elastic buckling analysis. Elastic buckling analysis was also used to obtain the eigen modes to represent the geometric imperfection distribution shape required for non-linear analysis. Ultimate strength of the foam supported steel plate elements was determined from a non-linear analysis. In this study, a plastic constitutive model with a Mises/Hill yield criteria and a perfect plasticity hardening rule was used in the non-linear analysis. RIKS solution techniques with a default convergence tolerance were used in the non-linear analysis.

As already investigated and verified by Pokharel and Mahendran (2004a), sandwich panels tested in the laboratory can be simulated by finite element analysis using a half-length model. All the slender plates tested in this study were therefore modelled and analysed using half-length models. Hence only half the length ($L/2$) of the panel was used to create and analyse the model using

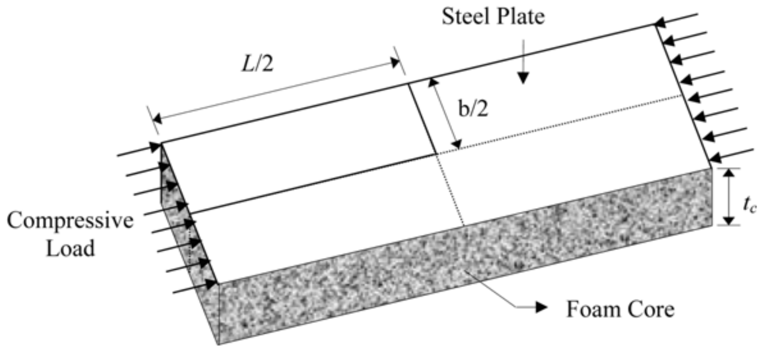

Fig. 4 Concept of half-length model

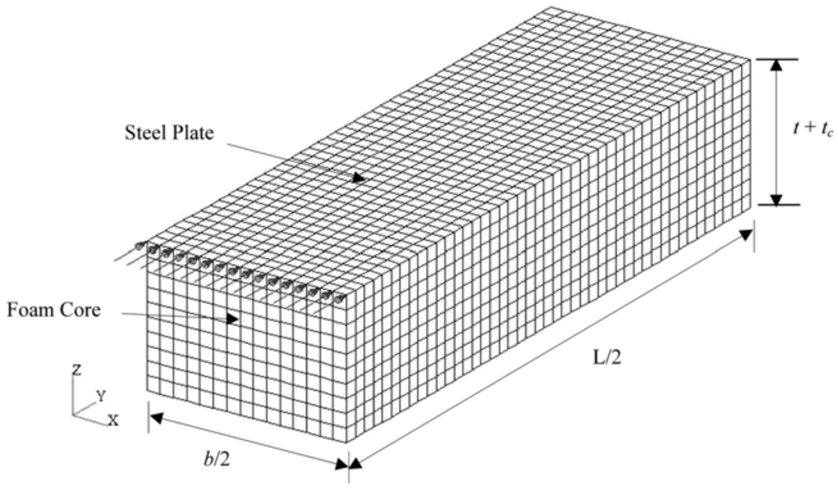

Fig. 5 Half-length model of foam supported steel plate

appropriate boundary conditions. Also, by using half the width ($b/2$), the half-length model was reduced to the quarter size of the full panel. As the full panel was reduced to quarter size, large number of elements with smaller sizes were used in finite element meshing that increased the level of accuracy of numerical results. Fig. 4 shows the concept and actual dimensions used in the half-length model. A constant foam thickness of 100 mm was used for all the FEA models as used in the experimental panels. Mesh sizes of 10×10 mm for steel faces and $10 \times 10 \times 5$ mm for foam core were used to obtain satisfactory results based on a convergence study. Fig. 5 shows the model geometry, mesh size and the loading pattern for the half-length model.

The half-length model was analysed using both elastic buckling and non-linear analyses. For the non-linear analysis, the first buckling mode obtained from the elastic buckling analysis was used as the geometric imperfection distribution shape and 10% of the plate thickness ($0.1t$) was used as the maximum imperfection magnitude. The ultimate stress of the foam-supported plate elements was obtained from the non-linear analyses. To validate the results obtained from the half-length model, some of the specimens were modelled and analysed using a full-length model. The differences in the results obtained from full-length and half-length models were insignificant with a maximum difference of 3%. This confirmed the adequacy of half-length model in simulating the behaviour of experimental panels. Therefore further analyses in this study were conducted using the half-length model with only half width to save on computational time.

4.2 Comparison of experimental and FEA results

The ultimate strengths obtained from the half-length models were than compared with the experimental results in Table 2. It must be noted here that the investigation was undertaken for foam supported plate elements with a wide range of *b/t* ratios (200 to 1000) to observe and fully understand the change in buckling behaviour of the foam supported steel plate elements with increasing *b/t* ratios. From Table 2, it can be observed that the ultimate strength results obtained from the half-length models agree reasonably well with the experimental results for all the test specimens including the very slender plates. This agreement can be confirmed further in Fig. 6 where a comparison of typical axial compression load versus axial shortening curves from FEA and experiments is presented.

Both experimental and FEA results indicated that the ultimate stress of the foam-supported steel plate elements is dependent on the *b/t* ratio of the plate elements. For plates with low *b/t* ratios, the ultimate stress is high because of the postbuckling strength. As the plate *b/t* ratio increases, the ultimate stress decreases gradually. However, when the *b/t* ratio is large (>500), the ultimate stress is very low and does not vary significantly even if the *b/t* ratio increases. For very high *b/t* ratios (>800), the ultimate stress remains almost constant.

As seen in Table 2, when the *b/t* ratio of plate element is in the range of 200 to 700, the ultimate stress is decreasing continually. But as the *b/t* ratio increases further (say 800 to 1000), there is no significant change in the ultimate stress as seen from the FEA results. It shows that the ultimate stress of foam-supported plate elements is independent of plate width and thickness for the plates with very high *b/t* ratios. The ultimate stress of very slender plates is always constant irrespective of the *b/t* ratio. This indicates that very slender plates fail due to wrinkling and the theoretical wrinkling stress can be evaluated by the wrinkling formula using Eq. (9). Using this formula, the

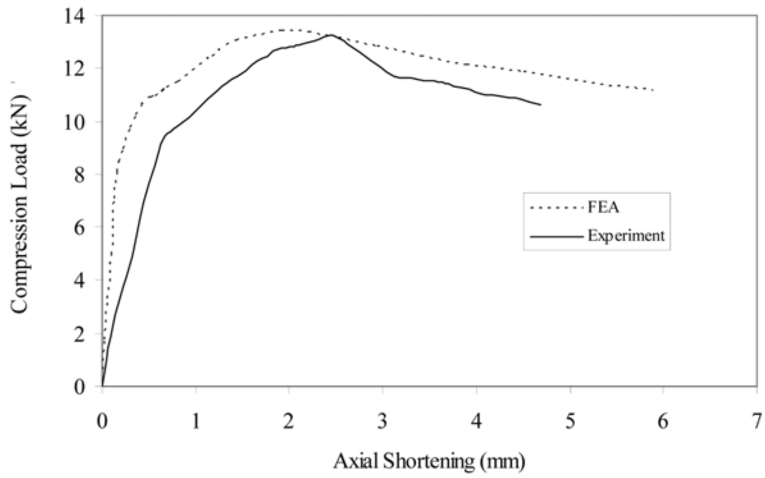

Fig. 6 Typical axial compression load versus axial shortening curve (b = 180 mm, t = 0.60 mm)

theoretical wrinkling stress of very slender plates was found to be 95.17 MPa for 0.42 mm thick G550 plate and 94.64 MPa for 0.60 mm thick G550 plate, respectively. In this investigation, it can be clearly observed that the ultimate stresses of the plate elements with a b/t ratio >800 are close to the above theoretical wrinkling stress values. Hence they are likely to fail by wrinkling. However, the ultimate stresses of the plate elements with b/t ratio less than 700 are more than the theoretical wrinkling stress. These observations indicate that although these plates ($b/t < 700$) exhibit either very little or no post buckling strength, they do not fail by wrinkling.

4.3 Half-wave buckle length model to represent real panels

As studied and verified by Pokharel and Mahendran (2004a), the experimental foam-supported plate elements do not necessarily represent the actual sandwich panels. The width of the foam core used in the experiment was made the same as the steel plate in order to simplify the testing procedure. However, the foam in real sandwich panels is continuous in the width direction. Hence, to study the buckling and ultimate strength behaviour of the realistic profiled sandwich panels with slender plates, the half-wave buckle length model was developed and analysed for a wide range of b/t ratios up to 2000. In this model, half of the half-wave buckle length was used to create the geometry of the model. In sandwich panels, the half-wave buckle length is very small due to the stiffening effect of the foam core. Therefore the half-wave buckle length model is significantly smaller than the half-length model. The half-wave buckle length model is very similar to the theoretical model based on the elastic half-space method (Pokharel and Mahendran 2004a). Fig. 7 shows the concept and typical dimensions of the half-wave buckle length model used in this study. As in the case of half-length model, the steel plate and foam core of half-wave buckle length model were modelled using S4R5 thin shell elements and C3D8 solid elements, respectively.

In order to create the half wave buckle length model, the single half-wave buckling length (a) has to be determined first. The length of the half-wave buckle length model, $a/2$, was found by varying $a/2$ using a series of elastic buckling analyses until the minimum eigenvalue and thus the buckling stress was obtained. The theoretical approach of determining the half-wave buckling length (a) is based on the energy method (Pokharel and Mahendran 2004a). A steel plate supported on an infinitely deep foam core represents the plate on an elastic foundation as considered in the energy method. Fig. 8 shows the model geometry, mesh size and the loading pattern for the half-wave buckle length model. Using this half-wave buckle length model, the buckling and ultimate stresses

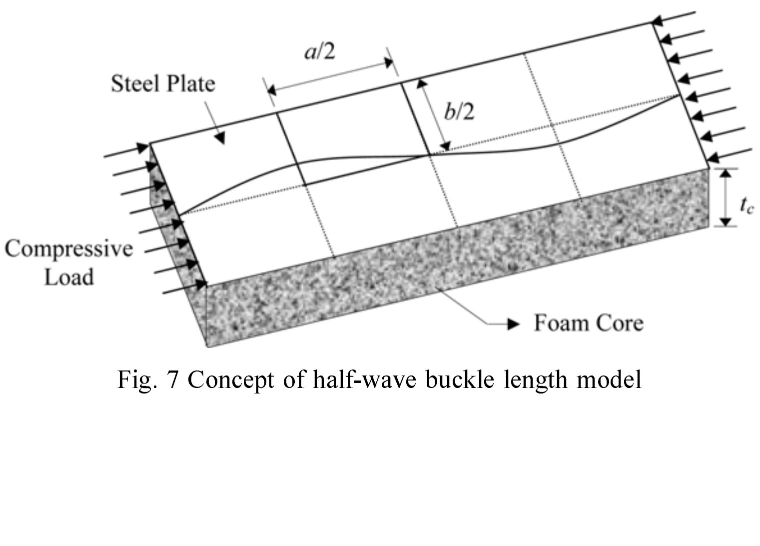

Fig. 7 Concept of half-wave buckle length model

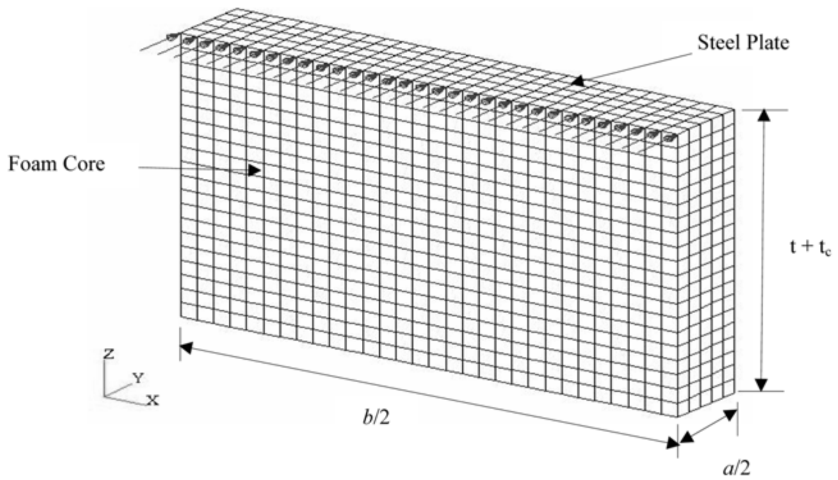

Fig. 8 Half-wave buckle length model of foam supported steel plate

Table 3 Buckling and ultimate stresses of 0.42 mm thick G550 grade steel from half-wave buckle length model

Plate width *b* (mm)	Plate thickness *t* (mm)	*b/t* ratio	Half-wave buckle length *a* (mm)	Buckling stress (MPa)	Ultimate stress (MPa)
80	0.42	190.5	16	113.39	188.69
100	0.42	238.1	17	106.67	160.95
130	0.42	309.5	17	102.18	132.38
150	0.42	357.1	17	100.63	122.70
170	0.42	404.8	17	99.56	115.24
190	0.42	452.4	17	98.85	110.48
210	0.42	500.0	18	98.09	108.57
250	0.42	595.2	18	97.38	102.86
290	0.42	690.5	18	96.95	100.00
340	0.42	809.5	18	96.63	97.14
380	0.42	904.8	18	96.47	96.67
420	0.42	1000.0	18	96.34	96.19
460	0.42	1095.2	18	96.26	95.24
500	0.42	1190.5	18	96.19	95.24
630	0.42	1500.0	18	96.06	95.24
840	0.42	2000.0	18	95.97	95.24

were evaluated using elastic buckling and non-linear analyses, respectively. The buckling and ultimate stress results obtained from the FEA based on the half-wave buckle length model for the foam-supported steel plate elements (0.42 mm thick G550 steel) with *b/t* ratio ranging from 200 to 2000 are presented in Table 3.

As seen in Table 3, when the *b/t* ratio of the foam-supported steel plate element is in the lower

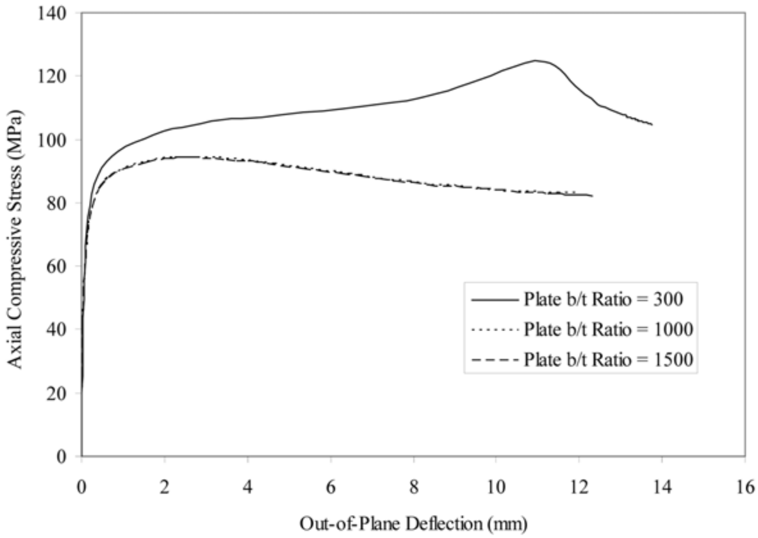

Fig. 9 Typical axial compressive stress versus out-of plane deflection

range (say < 400), the ultimate stress of the panel is higher than the buckling stress due to the presence of considerable postbuckling strength. When the *b/t* ratio increases, the postbuckling strength continually diminishes and the buckling and ultimate stresses are almost equal. For very slender plates, the ultimate stress is low and is almost a constant. It can be clearly observed that the ultimate stress of the plate elements with a *b/t* ratio > 1000 is constant (95.24 MPa) and is independent of the *b/t* ratio. This FEA ultimate stress value compares well with the theoretical wrinkling stress value of 95.17 MPa for 0.42 mm thick G550 steel plate. This obviously indicates the wrinkling failure of the foam-supported steel plate elements with very high *b/t* ratios. Fig. 9 shows the axial compressive stress versus out-of-plane deflection curves of foam-supported steel plate elements with *b/t* ratios 300, 1000 and 1500. This figure clearly indicates the considerable postbuckling strength for the compact plate (*b/t* ratio = 300) whereas no postbuckling strength for slender plates (*b/t* ratios = 1000 and 1500). Once again the results confirmed that the failure mode of very slender plates is dominated by wrinkling. However, for the plates with *b/t* ratio less than 1000, the ultimate stresses are higher than the wrinkling stresses, but there is no postbuckling strength.

Summarising the results of Table 3, foam-supported steel plate elements exhibit considerable postbuckling strength when the *b/t* ratio is less than 400. For *b/t* ratios from 400 to 1000, either little or no postbuckling strength can be observed, but the ultimate stresses are higher than the theoretical wrinkling stresses and the buckling and ultimate stresses are almost equal. However, for the *b/t* ratio more than 1000, the ultimate stresses are always constant and independent of the *b/t* ratio. This investigation therefore confirms that if the plate elements in fully profiled sandwich panels are very slender (*b/t* ratio more than 1000), their strength can be evaluated by the wrinkling formula. For the slender plate elements with *b/t* ratio less than 1000, the wrinkling formula will underestimate the strength of sandwich panels as the failure stress is higher than the theoretical wrinkling stress. The ranges of *b/t* ratios, for which there is either very little or no postbuckling strength but the failure stress (ultimate stress) is higher than the wrinkling stress can be termed as

Table 4 Buckling and ultimate stresses of 0.6 mm thick G550 grade steel plate from half-wave buckle length model

Plate width b (mm)	Plate thickness t (mm)	b/t ratio	Half-wave buckle length a (mm)	Buckling stress (MPa)	Ultimate stress (MPa)
120	0.60	200.0	23	110.00	155.42
150	0.60	250.0	24	104.33	141.89
180	0.60	300.0	24	101.30	124.72
210	0.60	350.0	24	99.52	114.00
240	0.60	400.0	24	98.40	107.33
270	0.60	450.0	24	97.65	103.33
300	0.60	500.0	25	97.11	102.67
360	0.60	600.0	25	96.39	96.67
420	0.60	700.0	25	95.99	95.33
480	0.60	800.0	25	95.73	95.00
540	0.60	900.0	25	95.55	94.67
600	0.60	1000.0	25	95.44	94.36
660	0.60	1100.0	25	95.35	94.00
720	0.60	1200.0	25	95.30	94.00
900	0.60	1500.0	25	95.17	94.00
1200	0.60	2000.0	25	95.10	94.00

the intermediate region. In this observation, the b/t ratios ranging from 400 to 1000, which neither shows significant postbuckling strength nor fail by wrinkling, can be considered as the intermediate region. Similar types of strength behaviour can be observed for other thicknesses of steels as can be seen in Table 4 for 0.60 mm thick G550 steel and Table 5 for 0.95 mm G550 steel, respectively.

In the initial intermediate region, the ultimate stress is higher than the theoretical wrinkling stress, but the difference decreases considerably in the latter part of the region (700 to 1000). Therefore the failure stress of the plates in this latter part can still be determined by using the wrinkling formula. However, for the plate element with b/t ratio less than 600, the failure stress is higher than the wrinkling stress, and the wrinkling formula can not be used to determine the strength of the panels in that region. Current effective width design rule (Eqs. (1) to (4)) is applicable only for the plate elements with b/t ratio less than 100 as already investigated by Pokharel and Mahendran (2003) using experimental and FEA studies. No design rule exists for the safe design of profiled sandwich panels with slender plate elements in this initial part of intermediate region. As the b/t ratios of most practical sandwich panels are in the range of 100 to 600, it is necessary to develop a design rule that can be used for the profiled sandwich panels with any b/t ratio up to 600. Based on the results from this study, a b/t ratio of 600 can be considered as a reasonable boundary to develop the new design rule for fully profiled sandwich panels. In order to provide a necessary design solution for sandwich panels, Pokharel and Mahendran (2004a) conducted an extensive series of experiments and finite element analyses and developed a new design rule that can be applied to the foam supported plate elements subject to local buckling effects with any plate b/t ratio in the local buckling and the initial part of intermediate regions. This new design rule as given in Eqs. (6), (7) and (8) was developed based on the local buckling, postbuckling and ultimate strength behaviour of

Table 5 Buckling and ultimate stresses of 0.95 mm thick G550 grade steel from half-wave buckle length model

Plate width b (mm)	Plate thickness t (mm)	b/t ratio	Half-wave buckle length a (mm)	Buckling stress (MPa)	Ultimate stress (MPa)
200	0.95	210.5	37	105.94	133.47
240	0.95	252.6	38	101.63	122.11
280	0.95	294.7	39	99.09	117.05
330	0.95	347.4	39	97.15	109.89
380	0.95	400.0	39	95.96	101.05
430	0.95	452.6	40	95.17	98.11
480	0.95	505.3	40	94.64	94.74
570	0.95	600.0	40	94.02	93.47
660	0.95	694.7	40	93.65	93.05
760	0.95	800.0	40	93.39	92.84
850	0.95	894.7	40	93.24	92.63
950	0.95	1000.0	40	93.12	92.42
1050	0.95	1105.3	40	93.03	92.21
1140	0.95	1200.0	40	92.97	92.21
1420	0.95	1494.7	40	92.85	92.21
1900	0.95	2000.0	40	92.79	92.21

foam supported steel plate elements as used in the fully profiled sandwich panels.

Finally, based on the observations of current experimental and finite element analyses results, it can be recommended that for the plate elements with b/t ratio more than 600, the wrinkling formula (Eqs. (9) and (10)) for flat faced sandwich panels can be used to determine the strength of fully profiled sandwich panels, although it is slightly conservative. If the b/t ratio of the plate element is less than 600, the ultimate strength of the panel should be determined by using the effective width formula given in Eqs. (6) to (8). Furthermore, as indicated by Pokharel and Mahendran (2004a), if the b/t ratio is less than 100, the strength of the plate can be determined by using the effective width formula developed by Davies and Hakmi (1990) as given in Eqs. (1) to (4) or by those developed by Pokharel and Mahendran (2004a) as given in Eqs. (6) to (8).

5. Conclusions

The bucking and ultimate strength behaviour of foam supported plate elements as used in fully profiled sandwich panels with a wide range of b/t ratios was investigated using a series of experiments and finite element analyses. The structural performance and failure modes of plate elements in the local buckling, wrinkling and the intermediate regions were fully investigated and the plate slenderness limits that separate the various regions were identified. Based on the buckling failures exhibited by the plate elements with increasing b/t ratios, suitable design methods were recommended to predict the ultimate failure strength of the fully profiled sandwich panels. It was recommended that the ultimate strength of the foam supported steel plate elements whose b/t ratios lie in the local buckling region and the initial part of the intermediate region (b/t < 600), can be determined by the effective width formula developed by Pokharel and Mahendran (2004a). On the other hand, if the plate slenderness ratio lies in the latter part of the intermediate region (b/t > 600) and the wrinkling region, the wrinkling formula developed for flat faced sandwich panels can be used.

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