Local buckling behaviour and design of profiled sandwich panels*

Narayan Pokharel[1] and Mahen Mahendran[2]
[1]PhD Student, [2]Professor
School of Civil Engineering, Queensland University of Technology, Brisbane, Queensland

SUMMARY: Fully profiled sandwich panels are commonly used in building structures, but are susceptible to local buckling failures. Current European design standard recommends the use of a modified effective width approach to include such local buckling effects in the design. Recent research has shown that this design method predicts unconservative strengths for panels with slender plates (high b/t ratios). The use of sandwich panels with high b/t ratios is very common in practice due to the increasing use of thinner and high strength steel plates. Therefore, a research project was conducted to investigate the local buckling behaviour of foam supported steel plate elements as used in fully profiled sandwich panels with a large range of b/t ratios (50 to 500) using experiments and finite element analyses (FEA). Both experimental and FEA results revealed the inadequacy of the current design rule for sandwich panels with slender plate elements. An improved design method was therefore developed based on the large amount of data obtained from validated FEA studies. Finally full-scale tests of fully profiled sandwich panels were undertaken to confirm the accuracy of the new design method. This paper presents an overview of these experimental and finite element analysis studies and design rule development followed by the results of the full-scale tests.

1 INTRODUCTION

In the past, sandwich panels have been commonly used in many aeronautical applications. Their use has now been extended to commercial and residential building construction due to their ability to improve their structural and thermal performance. Until recently, sandwich panel construction in Australia has been limited to cold-storage buildings due to lack of design methods and data. However, in recent times, they are increasingly used in building structures, particularly as roof and wall cladding systems.

The structural behaviour of a sandwich panel is based on a composite action of its three components, namely the two outer faces and the inner core.[1] Generally the faces of sandwich panels are made of very thin steel and are susceptible to various buckling failures under the action of compression, bending or their combinations. The buckling failure modes mainly depend on the types of steel faces used. Sandwich panels with fully profiled faces are subjected to local buckling failures (see Figure 1) and hence local buckling is the main design criterion for such panels.

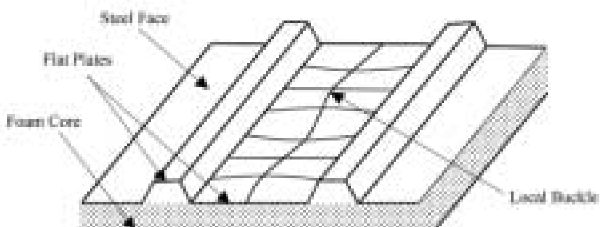


Figure 1 Local Buckling of Fully Profiled Sandwich Panels

Extensive research has been undertaken during the last decade to investigate the local buckling behaviour of plate elements in fully profiled sandwich panels and to develop rational design procedures. In Davies and Hakmi's[2,3] research, the local buckling phenomenon of fully profiled sandwich panels was treated in design by utilizing a modified effective width approach. The original effective width method[12] developed for plain plate elements was

* Paper S24/923 submitted15/3/2004
Paper accepted for publishing 30/6/2004

extended to the foam supported plate elements by using the concept of a modified buckling coefficient. This design method has now been included in a European design document.[4]

Past research on sandwich panels that was used to develop design rules has been based on thicker and lower grade steels and polyurethane or polyisocyanurate foam cores. Sandwich panels generally used in Australia comprise of thinner (0.42 mm) and high strength (minimum yield stress of 550 MPa and reduced ductility) steel faces and relatively thick polystyrene foam core which are bonded together using separate adhesives. Therefore there is a need to verify the applicability of European design recommendations[4] to Australian sandwich panels in order to develop confidence among the Australian manufacturers and designers.

Recent research[7] has indicated that the modified effective width approach[2] can be successfully used for plate elements with a low *b*/*t* ratio, but can not be extended to slender plates as it leads to unconservative strengths. The plate elements generally used in fully profiled sandwich panels are slender, and a safe design rule is not available. Therefore a research project was undertaken at Queensland University of Technology to investigate the local buckling behaviour of foam supported plate elements and develop a new design rule that can be used in the design of fully profiled sandwich panels with any practical *b*/*t* ratios of the plate elements.

The first stage of this research was based on a series of laboratory experiments and numerical analyses of 50 foam-supported steel plate elements made of thin high strength steel and polystyrene foam core covering a wide range of *b*/*t* ratios. Based on the experimental results and corresponding finite element analysis results, an improved design rule was developed for sandwich panels with any practical *b*/*t* ratios. In the second stage, full-scale experiments of fully profiled sandwich panels were undertaken to examine the accuracy of the new improved design rule. This paper presents a summary of the first stage of this research project, in particular the details of experiments, finite element analyses and design rules, and the details of full-scale tests, the results and comparisons with the design rules.

2 COMPRESSION TESTS OF FOAM SUPPORTED STEEL PLATES

Local buckling behaviour of sandwich panels with profiled steel faces was investigated experimentally by conducting compression tests on 50 steel plate elements (25 each for steel grades G550 and G250) supported by polystyrene foam cores. As the foam thickness has negligible effect on the buckling strengths[6], a constant thickness of 100 mm was used in the tests. To cover a large range of *b*/*t* ratios (between 50 to 500), thicknesses ranging from 0.4 to 1.0 mm and widths ranging from 50 to 200 mm were used for each grade of steel. The plate lengths were chosen as three times the width *b* plus 10 mm for clamping. The steel faces and foam were glued to each other by using a suitable adhesive. The initial imperfections of specimens relating to the flatness of steel plates were found to be minimal as was the case in most of the fully profiled sandwich panels. Details of the experimental program and test specimens are given in Table 1.

Table 1
Test Specimens and Experimental Program

Test Series	Plate Width b (mm)	G550 Steel Plates				G250 Steel Plates			
		Base Metal Thickness t (mm)	Measured f_y (MPa)	Measured E_f (GPa)	b/t Ratio	Base Metal Thickness t (mm)	Measured f_y (MPa)	Measured E_f (GPa)	b/t Ratio
1	50	0.95	637	226	52.6	0.93	326	216	53.8
2	50	0.80	656	230	62.5	0.73	345	217	68.5
3	50	0.60	682	235	83.3	0.54	360	218	92.6
4	50	0.42	726	239	119.0	0.39	368	220	128.2
5	80	0.95	637	226	84.2	0.93	326	216	86.0
6	80	0.80	656	230	100.0	0.73	345	217	109.6
7	80	0.60	682	235	133.3	0.54	360	218	148.1
8	80	0.42	726	239	190.5	0.39	368	220	205.1
9	100	0.95	637	226	105.3	0.93	326	216	107.5

Test Series	Plate Width b (mm)	G550 Steel Plates				G250 Steel Plates			
		Base Metal Thick-ness t (mm)	Measured		b/t Ratio	Base Metal Thick-ness t (mm)	Measured		b/t Ratio
			f_y (MPa)	E_f (GPa)			f_y (MPa)	E_f (GPa)	
10	100	0.80	656	230	125.0	0.73	345	217	137.0
11	100	0.60	682	235	166.7	0.54	360	218	185.2
12	100	0.42	726	239	238.1	0.39	368	220	256.4
13	120	0.95	637	226	126.3	0.93	326	216	129.0
14	120	0.80	656	230	150.0	0.73	345	217	164.4
15	120	0.60	682	235	200.0	0.54	360	218	222.2
16	150	0.95	637	226	157.9	0.93	326	216	161.3
17	150	0.80	656	230	187.5	0.73	345	217	205.5
18	150	0.60	682	235	250.0	0.54	360	218	277.8
19	150	0.42	726	239	357.1	0.39	368	220	384.6
20	180	0.60	682	235	300.0	0.54	360	218	333.3
21	180	0.42	726	239	428.6	0.39	368	220	461.5
22	200	0.95	637	226	210.5	0.93	326	216	215.1
23	200	0.80	656	230	250.0	0.73	345	217	274.0
24	200	0.60	682	235	333.3	0.54	360	218	370.4
25	200	0.42	726	239	476.2	0.39	368	220	512.8
Foam Properties: Ec = 3.8 MPa, Gc = 1.76 MPa, νc = 0.08									

A specially constructed test rig was used to hold the test specimen with two vertical clamps allowing the vertical displacement and free rotation at the longitudinal edges, as required for the simply supported conditions. The vertical supports were adjustable in both horizontal and vertical directions to accommodate the required plate width and length, respectively. The test specimens were placed in the test rig between two loading blocks and loaded in compression to failure using a Tinius Olsen Testing Machine. It is to be noted that the compression load was applied to the steel plate element only and not to the foam core. The arrangement of the test set-up is shown in Figure 2.

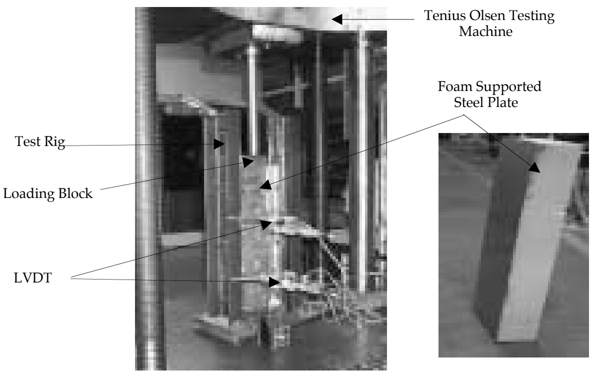
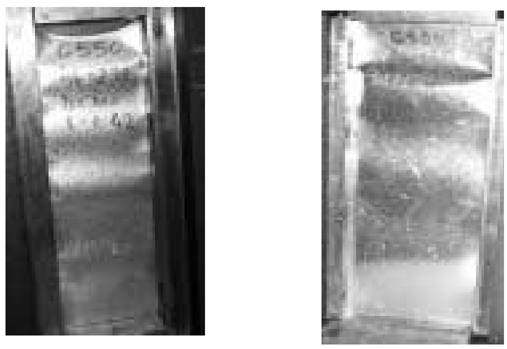


Figure 2 Test Set-Up

Tests showed that all the test specimens first buckled locally as shown in Figure 3a, then developed postbuckling strength, reached the ultimate load and collapsed through the formation of a local plastic mechanism as shown in Figure 3b. Experiments on foam supported steel plate elements confirmed that the foam core reduced the half wave buckle length ($a < b$) and produced many half wave buckles within the test specimen. This led to increased buckling strength. The buckling and ultimate strength results obtained from the experimental investigation were used to validate the finite element models as described in Section 4. Further details of the experimental investigation are given in Pokharel and Mahendran.[7]



(a) Local Buckling (b) Local Plastic Mechanism

Figure 3 Typical Failure Modes

3 REVIEW OF CURRENT DESIGN RULES

European recommendation for sandwich panels, Part 1: design[4] recommends that if the outermost plate element in a fully profiled sandwich panel is in compression and the width to thickness (b/t) ratio exceeds the limit given in Equation 1, it will be subjected to local buckling effects and due regard should be given to this phenomenon while designing the sandwich panels.

$$\frac{b}{t} = 1.27\sqrt{\frac{E_f}{f_y}} \tag{1}$$

where f_y = yield stress and E_f = Young's modulus of steel face. In practice, the b/t ratio of the flat part of the steel plate element in a profiled sandwich panel is always higher than the limit specified by Equation 1. Hence, it is obvious that profiled sandwich panels are always susceptible to local buckling effects when subjected to compression, bending or their combinations. This local buckling phenomenon causes loss of stiffness of the panel. However, considerable postbuckling strength will be developed due to redistribution of stresses after local buckling. This implies that local buckling and postbuckling phenomena are very important in the design of profiled sandwich panels.

The compression strength f_{Fc} of the profiled faces depends on the yield stress of the face material f_y, width to thickness ratio (b/t) of the most stressed plane part of the profile, the compressive and shear stiffness of the core material, initial imperfection caused by the face, foam core, and the bond between face and core.[4] It can be evaluated using the following formula:

$$f_{Fc} = \frac{b_{eff}}{b} f_y \tag{2}$$

where b_{eff} is the effective width of plane parts of a face profile and determined by using the effective width approach given in Equations 3 and 4.

$$\frac{b_{eff}}{b} = \frac{1}{\lambda}\left[1 - \frac{0.22}{\lambda}\right] \tag{3}$$

$$\lambda = 1.052\left[\frac{b}{t}\right]\sqrt{\frac{f_y}{E_f K}} \tag{4}$$

where K is the buckling coefficient.

Equations 3 and 4 are the original effective width formulae for flat plate elements without foam core. They were developed by Winter[12] based on the local buckling and postbuckling strength of cold-formed steel plates and sections. Davies and Hakmi[2] extended this effective width approach to fully profiled sandwich panels by using a modified buckling coefficient K in Equation 4. If the effect of foam core is ignored, the value of K is 4.0. However, if the support of the core is utilised, a higher value of buckling coefficient K should be used. K for profiled sandwich panels can be determined by using a theoretical approach based on the elastic half-space method. This is given by the following equation[2]:

$$K = \left[\frac{1}{\phi} + n^2\phi\right]^2 + R\phi\left[1 + n^2\phi^2\right]^{\frac{1}{2}} \tag{5}$$

where ϕ is the ratio of half-wave buckle length a to the width of the plate b ($\phi = a/b$) and R is the stiffness parameter which models the influence of composite action between the steel faces and foam core.

However, many researchers have developed explicit mathematical equations to determine the enhanced buckling coefficient K for the foam supported steel plates. Davies and Hakmi[2] proposed the following equation to determine K for the design of sandwich panels.

$$K = [16 + 11.8R + 0.055R^2]^{1/2} \quad \text{with}$$

$$R = \frac{12(1-\nu_f^{\,2})\sqrt{E_c G_c}}{\pi^3 E_f}\left[\frac{b}{t}\right]^3 \tag{6}$$

where E_c and G_c are the elastic modulus and shear modulus of the core, respectively, E_f and ν_f are the elastic modulus and Poisson's ratio of the steel face, respectively. Davies and Hakmi[2] indicated that this equation is accurate for a range of R from 0 to 200.

Mahendran and Jeevaharan[5] conducted a series of tests and finite element analyses on foam supported steel plate elements to investigate the local buckling behaviour. From this study, they proposed Equation 7 for K that can be applied for higher values of R up to 600.

$$K = [16 + 4.76R^{1.29}]^{1/2} \quad \text{with}$$

$$R = \frac{12(1-\nu_f^{\,2})\sqrt{E_c G_c}}{\pi^3 E_f}\left[\frac{b}{t}\right]^3 \tag{7}$$

In the current European Recommendations for Sandwich Panels, Part I: Design[4], Equation 8 with the following R value has been recommended for predicting the value of K.

$$K = [16 + 7R + 0.02R^2]^{1/2} \quad \text{with}$$

$$R = 0.35\frac{\sqrt{E_c G_c}}{E_f}\left[\frac{b}{t}\right]^3 \qquad (8)$$

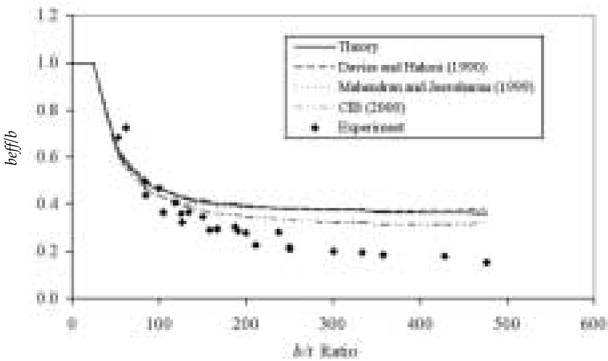
This equation was obtained by replacing R with an empirical reduction factor of $0.6R$ in Equation 6 as recommended by Davies and Hakmi.[2]

It is clear from the above discussion that, in the current design method, the effective width b_{eff} of the plane parts of a face profile can be determined from Equations 3 and 4 using the enhanced buckling coefficient K proposed by Davies and Hakmi[2], Mahendran and Jeevaharan[5], CIB[4] and a theoretical approach based on an energy method. In this approach, no distinction is made between the ductile low strength steels (G250) and the less ductile high strength steels (G550). Therefore Equations 3 and 4 can be used for both G250 and G550 steels of all thicknesses.

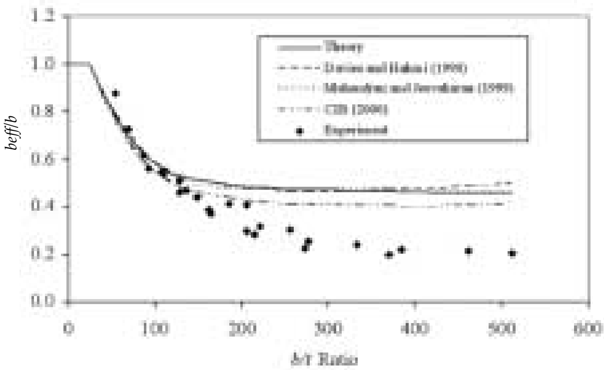
However, Clause 1.5.1.5 (b) of AS/NZS 4600[11] recommends that the yield stress of G550 grade steels with a thickness less than 0.9 mm should be multiplied by a reduction factor of 0.75 for design purposes as these steels do not satisfy the required ductility criteria. A recent study conducted by Yang and Hancock[13] showed that the predicted strength results for G550 steel columns based on 75% of yield stress ($0.75f_y$) are too conservative. They therefore recommended that a higher reduction factor of 0.90 is used for G550 steels with a thickness less than 0.9 mm. Since Yang and Hancock's[13] research was not for foam-supported steel plates, the accuracy of their recommendation to foam supported steel plate elements is not known. However, it was considered appropriate to use a reduction factor to allow for possible strength reduction in thinner G550 steel plates. In this experimental study, effective widths were calculated based on $0.9f_y$ for G550 grade steels with a thickness less than 0.9 mm to investigate this further. For G550 grade steels with a thickness more than 0.9 mm and for all thicknesses of G250 grade steel actual f_y values were used.

The ultimate stress results obtained from the experiments on foam-supported steel plates can also be converted to ratios of effective width b_{eff} to plate width b in order to compare with other results. This ratio was taken as the ultimate stress of the foam-supported steel plates divided by the yield stress f_y. Here also a reduced yield stress, $0.9f_y$, was used for G550 grade steels with a thickness less than 0.9 mm.

Effective widths evaluated from the various different design formulae listed above together with the experimental results are plotted against the b/t ratios in Figures 4 (a) and (b) for G550 and G250 steel plates, respectively. It can be observed from Figures 4 (a) and (b) that the effective widths (b_{eff}) evaluated from Equations 3 and 4 using K values predicted by theory and different buckling formulae agreed reasonably well with the experimental results for low b/t ratios (< 100). However, for higher b/t ratios, all the formulae predicted very high effective width values compared with the experimental results, i.e. unconservative. Hence experimental results indicated that none of the formulae could estimate reasonable values of effective width for slender plates with high b/t ratios (> 100).



(a) G550 Steel Plates



(b) G250 Steel Plates

Figure 4 Effective Widths of Steel Plates supported by Foam Core

Increasing use of thinner steel plates in recent times has led to the use of slender plates in sandwich panel manufacturing. In the Australian sandwich panel construction, the b/t ratio of plate elements can be as large as 600.[5] This study clearly indicates the inadequacy of the current design rules and the need for improved design rules for slender plates. To investigate this behaviour further, finite element analyses were conducted and their details are given in the following section.

4 FINITE ELEMENT MODELLING AND ANALYSIS

The finite element program ABAQUS was used to investigate the local buckling behaviour of profiled sandwich panels and MSC PATRAN was used as pre- and post-processors. The steel plate was modelled using S4R5 three dimensional thin shell elements with four nodes and five degrees of freedom per node. The foam core was modelled using C3D8 three dimensional solid (continuum) elements with eight nodes and three degrees of freedom per node. Since there was no relative movement between the steel faces and foam core, they were modelled as a single unit.

Measured material properties of polystyrene foam and steel faces were used in the analysis.[7] They are E_c = 3.8 MPa, G_c = 1.76 MPa, ν_c = 0.08 for foam whereas the values for both G550 and G250 grades of steels are given in Table 1. Poisson's ratio of steel was assumed to be ν = 0.3. Both materials were considered to be isotropic. Two different finite element models, a half-length model to compare with the experimental results and a half-wave buckle length model to simulate real conditions of the sandwich panels used in building structures, were used.

It is important that appropriate geometric imperfections and residual stresses are introduced in a finite element model while undertaking a non-linear analysis to simulate the true structural behaviour. However, residual stresses were not considered in the analysis of foam supported steel plate elements considered in this study as they did not involve cold-forming or welding of a section or similar fabrication/manufacturing process capable of producing higher residual stresses. In the case of geometric imperfections, the mode shape based on the lowest eigenmode is sufficient to adequately characterize the most influential geometric imperfections, and this is considered an acceptable conservative approach.[9] Therefore, in the non-linear analyses of this study, the mode shape of the first buckling mode with a maximum geometric imperfection magnitude of 10% of the plate thickness (0.1t) was used.

4.1 Half-Length Model

To simulate the foam-supported steel plate elements tested in the laboratory, a half-length model with only a half width was used with appropriate boundary conditions including that of symmetry. On the basis of a convergence study, 10 mm square mesh surface elements for steel plate and $10 \times 10 \times 5$ mm solid elements for the foam core were used. A constant foam thickness of 100 mm was used to simulate the experimental conditions. Figure 5 shows the model geometry, mesh size and the loading pattern for half-length models. Appropriate boundary conditions were applied only to the steel face at the loading end and one of the longitudinal edges to simulate the experiments whereas symmetric boundary conditions were applied to the entire surface (steel faces and foam core) along both the longitudinal direction and across the width.

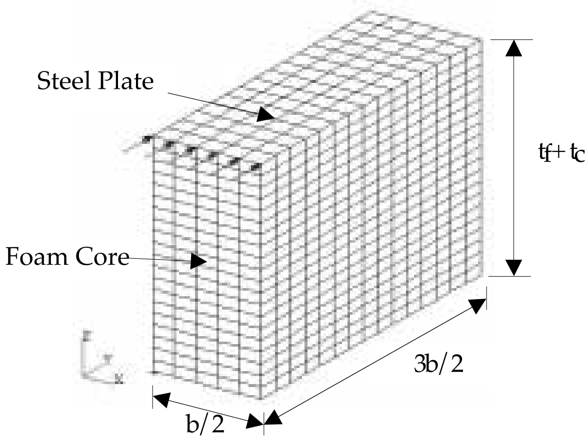
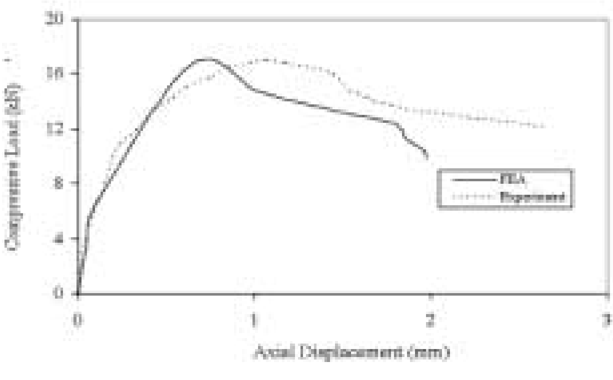
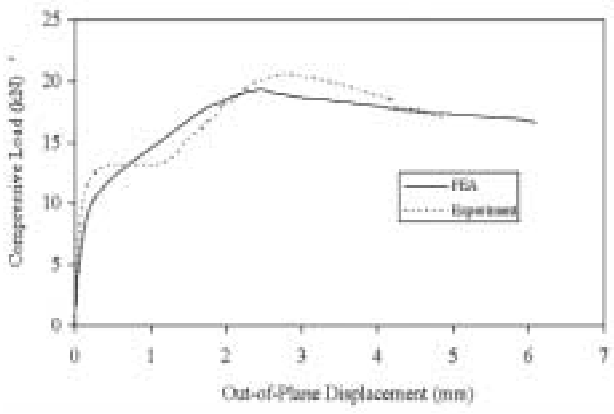


Figure 5 Half-Length Model of Foam Supported Steel Plate

Elastic buckling and ultimate loads were obtained from buckling and non-linear analyses, respectively. These results were compared with the corresponding experimental results. Figure 6 presents the comparison of typical load-deflection curves from the FEA and the experimental results. The results from the FEA and the experiments agreed reasonably well for both G550 and G250 steel plates. The mean values of the ratio of the FEA and the experimental buckling and ultimate stresses were found to be 1.00 and 0.94, respectively, for the G550 steel plates and 1.05 and 0.93, respectively, for the G250 steel plates. The corresponding coefficients of variation (COV) were 0.06 and 0.11, respectively, for the G550 steel plates and 0.08 and 0.12, respectively, for the G250 steel plates. These comparisons confirmed that half-length models can be successfully used to represent local buckling behaviour of experimental panels. Further details of this FEA investigation and the results are given in Pokharel and Mahendran.[8]



(a) Compressive Load vs Axial Displacement (G250, b = 200 mm, t = 0.93 mm)



(b) Compressive Load vs Out-of-Plane Displacement
(G550, b = 100 mm, t = 0.95 mm)

Figure 6 Comparison of Typical Load-Deflection Curves from FEA and Experiments

Since G550 steel plates with different thicknesses (0.42, 0.60, 0.80 and 0.95 mm) were used in this study, it was considered useful to investigate the ultimate strength behaviour of foam supported plate elements made of G550 steels. The mean values of the ratio of buckling and ultimate stresses from finite element analyses and experiments (FEA/Expt.) are compared in Table 2 for different thicknesses of G250 and G550 grade steels.

Table 2
Comparison of Results for Different Thicknesses of Steel

Steel Grade	Base Metal Thickness t (mm)	Mean of Buckling Stress Ratio FEA/Expt.	Mean of Ultimate Stress Ratio FEA/Expt.
G550	0.95	1.01	0.90
	0.80	0.96	0.86
	0.60	1.02	0.97
	0.42	1.01	1.05
G250	0.93	1.04	0.90
	0.73	0.99	0.90
	0.54	1.08	0.93
	0.39	1.09	1.01

As seen from the table, the ratios of the ultimate stresses from the FEA and the experiments were about the same (0.9) for thicker G550 steels (0.95 and 0.8 mm) and most of the G250 steels. However, the ultimate stress ratio increases as the G550 steel thickness decreases with a higher ratio of 1.05 for 0.42 mm G550 steel. This means that experimental strengths are less than the expected values for thinner G550 steels. Since the FEA did not include the possible strength reductions that could occur for thinner G550 steels with reduced ductility, the ultimate stress ratio is likely to increase with reducing thickness of G550 steel and will be greater than 1 for very thin steels. This can be seen in the results in Table 2. All of these observations therefore appear to confirm the AS/NZS 4600[11] requirement and Yang and Hancock's[13] recommendation of using a reduced yield stress ($0.75f_y$, $0.9f_y$) for G550 steels with a thickness less than 0.9 mm in predicting the member strengths. As expected there is no such observation with buckling stress ratios (see Table 2).

As mentioned earlier, the finite element model developed in this research is capable of simulating the local buckling behaviour of foam supported steel plate elements. However, it appears to overestimate the strength of thinner G550 steel plate elements compared with thicker G550 steels and most G250 steels. The reasons for this are given in Section 3, and the problem can be rectified by using a reduced yield stress. Therefore in this research it was assumed that further research is not required to investigate the reduction in ultimate strengths of members made of thinner G550 steels if a reduced yield stress $0.9f_y$ is used as recommended by Yang and Hancock.[13] The research was then continued using finite element models of both low and high strength steels. The finite element model does not simulate the behaviour of thinner G550 steels with reduced ductility, but is considered acceptable as the aim of this research was to study the effect of plate slenderness on the strength of sandwich panels and not the effect of reduced ductility of G550 steels on the ultimate strength behaviour.

4.2 Half-Wave Buckle Length Model

The foam-supported steel plate elements used in the experiments do not represent exactly those in practical sandwich panels.[8] However, validation of the half-length model by comparing its results with the experimental results provided the confidence in using the FEA model for simulating local buckling behaviour. The half-wave buckle length model matches the theoretical model used to develop the buckling stress formula based on the elastic half space method. Hence a single half-wave buckle was modelled with appropriate boundary conditions including that of symmetry. A mesh with 5 mm square surface elements for the steel plate and $5 \times 5 \times 5$ mm solid elements for the foam core was found appropriate and used for the half-wave buckle length models.

Appropriate boundary conditions were applied to the entire surface along all four sides. The length of the half-wave buckle model $a/2$ was found by varying $a/2$ using a series of elastic buckling analyses until the minimum buckling stress was obtained. The width of the model was $b/2$ (half the plate width), the length $a/2$, and the sum of the steel thickness t and a constant foam thickness of 100 mm was adopted. The model geometry and the mesh size used in the analyses are shown in Figure 7. The critical buckling load was obtained from elastic buckling analyses whereas the ultimate failure load was obtained from non-linear analyses.

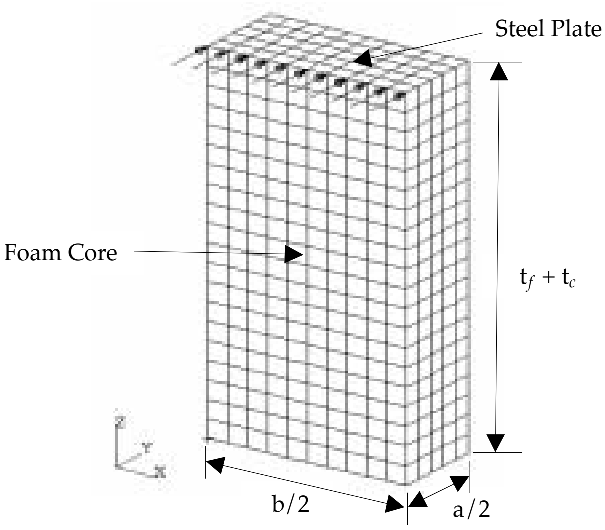


Figure 7 Half-Wave Buckle Length Model of the Foam supported Steel Plate

The critical buckling loads were compared with the theoretical results. The mean and COV of the ratio of the buckling loads from the FEA and theory were found to be 0.97 and 0.01, respectively, for the G550 steel plates, and 0.90 and 0.03, respectively, for the G250 steel plates. These close comparisons confirmed that the half-wave buckle length model can be successfully used to model the local buckling behaviour of steel plate elements in fully profiled sandwich panels to develop a new design rule. Further details of the numerical analyses and the results are given in Pokharel and Mahendran.[8]

5 NEW DESIGN RULE

From the FEA and experimental findings, it can be concluded that the current effective width approach can not be extended to the sandwich panels with slender plates in its present form. New improved design formulae have to be developed based on the finite element analysis results to estimate accurate values of effective widths that can be used for design purposes. To achieve this objective, the FEA results for all the specimens were evaluated and further FEA were undertaken to include b/t ratios from 30 to 600. Based on these FEA results, an improved design equation has been formulated as described next.

Effective width b_{eff} is considered as a particular width of the foam supported steel plate which just buckles when the compressive stress reaches the yield point of the steel. Using this assumption, b_{eff} can be determined using the following formula[14]:

$$\sigma_{cr} = f_y = \frac{K\pi^2 E_f}{12(1-\nu_f^{\,2})(b_{eff}/t)^2} \tag{9}$$

$$b_{eff} = \sqrt{K}.\sqrt{\frac{\pi^2}{12(1-\nu_f^{\,2})}}.t.\sqrt{\frac{E_f}{f_y}} = \sqrt{K}Ct\sqrt{\frac{E_f}{f_y}} \tag{10}$$

where $C = \sqrt{\dfrac{\pi^2}{12(1-\nu_f^{\,2})}} = 0.95$ (assuming $\nu_f = 0.3$) (11)

Before buckling, the width of the plate is fully effective and hence the critical buckling stress can be determined by using the full width b as follows:

$$\sigma_{cr} = \frac{K\pi^2 E_f}{12(1-\nu_f^{\,2})(b/t)^2} \tag{12}$$

$$b = \sqrt{K}.\sqrt{\frac{\pi^2}{12(1-\nu_f^{\,2})}}.t.\sqrt{\frac{E_f}{\sigma_{cr}}} = \sqrt{K}Ct\sqrt{\frac{E_f}{\sigma_{cr}}} \tag{13}$$

From Equations 10 and 13, the relationship between b_{eff} and b can be established as:

$$\frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{f_y}} \tag{14}$$

Equations 10 and 11 are the von Karman formulae for the design of stiffened elements developed in 1932. However, experimental investigations by Sechler[10] and Winter[12] showed that the term C used in Equation 10 depends primarily on the non-dimensional parameter γ expressed in the following way[14]:

$$\gamma = \sqrt{\frac{E_f}{f_y}}\left(\frac{t}{b}\right) \tag{15}$$

From Equation 10, the term C can be rewritten as:

$$C = \frac{b_{eff}}{t}\sqrt{\frac{f_y}{KE_f}} \tag{16}$$

From the finite element analysis conducted in this research, effective widths b_{eff} of foam supported plate elements were determined based on the ultimate stresses. Using Equation 16, the term C was evaluated for all the specimens considered. The corresponding non-dimensional parameter γ was determined using Equation 15. As the FEA results did not simulate the ductility characteristics of thinner G550 grade steels (t < 0.9 mm), the actual value of the yield stress f_y was used in all the calculations instead of $0.9f_y$. It was assumed that lower ultimate strengths of thinner G550 steel compression members could be separately dealt with by using $0.9f_y$ in the design calculations. A graph was plotted to establish the relationship between C and γ and the following equation was developed for the parameter C based on the FEA results.

$$C = 0.322(1 + 7.32\gamma - 11.48\gamma^2 + 4.59\gamma^3) \quad (17)$$

Substituting the value of γ into Equation 17,

$$C = 0.322(1 + 7.32\left(\frac{t}{b}\right)\left(\frac{E_f}{f_y}\right)^{1/2} - 11.48\left(\frac{t}{b}\right)^2\left(\frac{E_f}{f_y}\right) + 4.59\left(\frac{t}{b}\right)^3\left(\frac{E_f}{f_y}\right)^{3/2} \quad (18)$$

By substituting the value of C in Equation 10, a modified formula for computing the effective width b_{eff} for foam supported plate elements can be obtained.

$$\rho = \frac{b_{eff}}{b} = \frac{0.34}{\lambda}\left[1 + \frac{7.71}{\beta} - \frac{12.72}{\beta^2} + \frac{5.35}{\beta^3}\right] \quad (19)$$

where λ is the same as in Equation 4 and β is expressed as:

$$\beta = \lambda\sqrt{K} \quad (20)$$

The buckling coefficient K can be evaluated either using Equation 6 proposed by Davies and Hakmi[2] or Equation 7 proposed by Mahendran and Jeevaharan.[5] This new effective width formula (Equation 19) can be used for a wider range of b/t ratios from very compact to very slender ($b/t \le 600$) foam supported steel plate elements. To investigate the accuracy of the new design rules further, a series of full scale tests of fully profiled sandwich panels subjected to wind pressure loading was conducted. Details of the experimental procedure and the results are given in the next section.

6 FULL-SCALE TESTS OF FULLY PROFILED SANDWICH PANELS

6.1 Test Specimens and Test Procedure

Sandwich panels made from thin cold-formed steel faces and a polystyrene foam core bonded together using separate adhesives were used in this study. The polystyrene foam used in the panels was SL grade (density 13.5 kg/m^3). The top steel face was profiled whereas the bottom steel face was flat. Two different types of sandwich panels (Type A and Type B) were used in the experimental investigation as shown in Figure 8. In Type A panels, thickness of top and bottom steel faces were 0.42 mm (G550) and 0.60 mm (G300), respectively, whereas in Type B panels, they were 0.42 mm (G550) and 0.4 mm (G250), respectively. A foam core with a constant depth of 50 mm was used in all the panels. The spans of Type A panels were 2200, 2800 and 3300 mm with a constant width of 855 mm whereas the spans of Type B panels were 2200, 2550 and 2800 mm with a constant width of 466 mm.

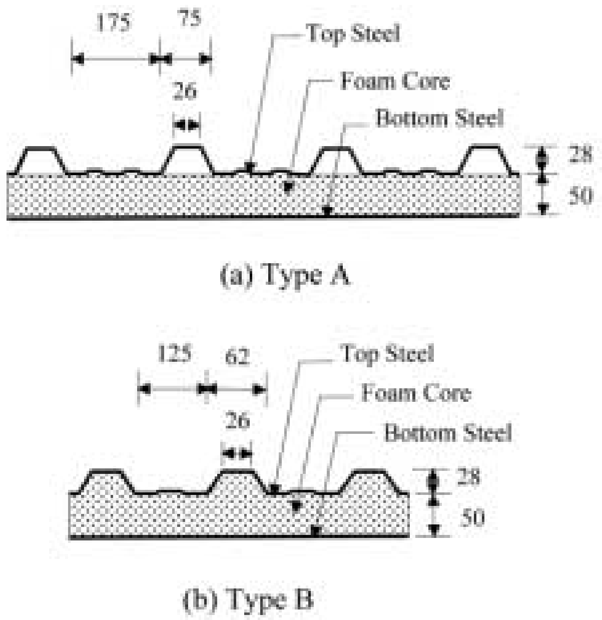


Figure 8 Types of Tested Profiled Sandwich Panels

The bending tests of six full-scale profiled sandwich panels (3 for each type) subjected to wind pressure loading were conducted in the Structural Laboratory. A large vacuum chamber (rectangular air box) was used to simulate a uniformly distributed transverse wind pressure loading on the underside of the sandwich panels. This arrangement produced a compressive stress in the top steel face and a tensile stress in the bottom steel face. The top profiled face in compression was thus subjected to a local buckling failure. Figure 9 shows a detailed schematic diagram of the test set-up including the vacuum chamber and the test panel. Test panels were simply supported over 70 mm wide rectangular hollow section (RHS) beams and were not restrained by the timber casing. Once the panel was positioned in the vacuum chamber, a polythene sheet was placed loosely over the panel. A photograph showing the test set-up is given in Figure 10. A vacuum pump was used to create a suction pressure in the chamber. The pressure applied to the panel was increased slowly until the panel collapsed due to bending and the failure pressure was measured using a pressure transducer.

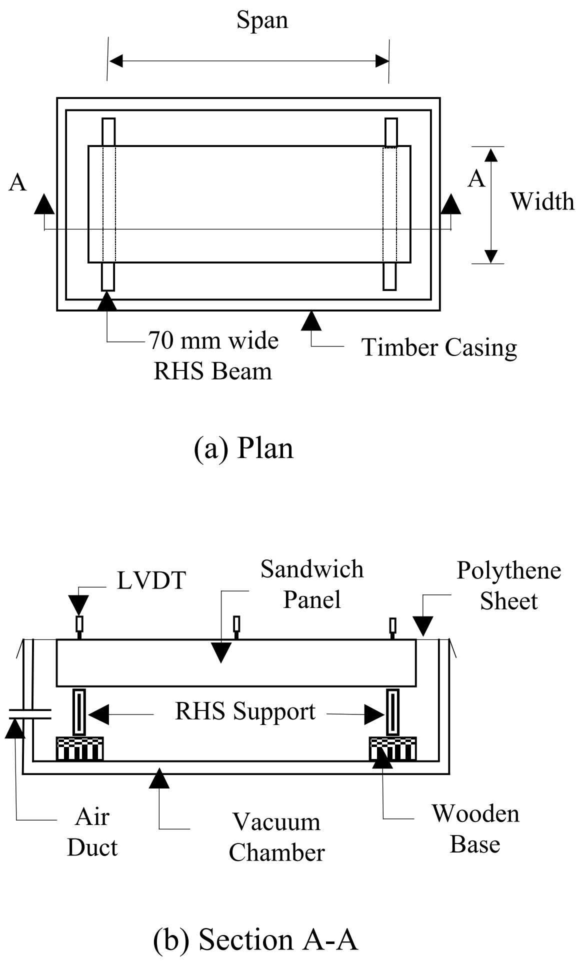


Figure 9 Schematic Diagram of Test Arrangement

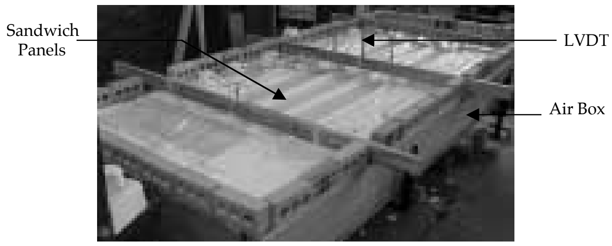


Figure 10 Test Set-Up

For all the panels tested, bending failure occurred in the vicinity of the midspan, that is, at the location of greater bending moment. The top profiled face, which was subjected to a compressive stress, first buckled locally, then developed postbuckling strength, and collapsed when it reached the ultimate pressure. Figure 11 shows the typical failure mode of the panel after the test. The measured experimental failure pressure was compared with the theoretically calculated value using the new design rule (Equations 19 and 20) and the CIB[4] design rule (Equations 3, 4 and 8). The experimentally measured values of the mechanical properties of the steel faces and the foam core were used (see Table 1) to calculate the theoretical value. A reduction factor of 0.9 was applied to the yield stress for the 0.42 mm thick G550 steel when calculating the effective widths as recommended by Yang and Hancock.[13]

Figure 11 Typical Failure Mode of Tested Sandwich Panel

6.2 Method of Predicting the Failure Pressures using Design Rules

To predict the failure pressure of the sandwich panels, it is necessary to determine the effective second moment of area of the panels, which is based on the effective widths of the compressive steel face (profiled face) of the section. For the Type A sandwich panels, the effective widths of the unsupported profiled ridge plates were evaluated using the standard effective width formula for plain plates where $K = 4.0$. The effective widths of the foam supported flat plates were evaluated using both the current CIB[4] design rule (Equations 3, 4 and 8) and the new design rule (Equations 19 and 20). For Type B sandwich panels, the effective widths of the plate elements for both the profile ridge and the flat plates were evaluated using the latter method as both of them were fully supported by the foam core.

It is necessary to consider the effect of the stress gradient when calculating the effective width of the inclined steel plate elements of the profiled ridge. The stress gradient increases the effective width of the plate elements by enhancing the buckling coefficient of the foam-supported steel plate elements. AS/NZS 4600[11] has recommended a formula to determine the increased buckling coefficient K_{inc} for the inclined plate elements without any foam support in the following form.

$$K_{inc} = 4 + 2(1-\Psi)^3 + 2(1-\Psi) \tag{21}$$

where ψ is the ratio of the stresses at the ends of the inclined plate element. The first term in Equation 21 is the normal buckling coefficient K of the plate element with simply supported boundary conditions ($K = 4.0$). As the inclined plate elements in the Type A sandwich panels are not supported by foam core, Equation 21 recommended by AS/NZS 4600 was directly applied to calculate the increased buckling coefficient K_{inc} of this element in the sandwich panel. However, the inclined plate elements in the Type B sandwich panels are supported by foam core and AS/NZS 4600 does not recommend any formula to calculate the increased value K_{inc} for such plate elements. The following formula was adopted to calculate the increased value of K_{inc} for the foam-supported steel plate elements.

$$K_{inc} = K + 2(1-\Psi)^3 + 2(1-\Psi) \tag{22}$$

where K is the normal buckling coefficient of the foam-supported steel plate element (more than 4.0 because of the composite action between the foam and the steel). The normal value of K for the foam-supported steel plate elements can be determined by using Equations 6 or 7.

Based on the effective widths of the top profiled face and the full width of the bottom face which is in tension, an effective second moment of area (I_{eff}) was calculated. Figure 12 shows the typical reduced cross-section used in calculating the effective widths. The small flat parts on both edges along the length of the panel were ignored while calculating the effective second moment of area as their effective widths are very small and they do not contribute much to the total strength of the panel. The failure pressure load w_u was then determined by equating the applied bending moment to the moment of resistance of the reduced cross-section as given in Equations 23 and 24.

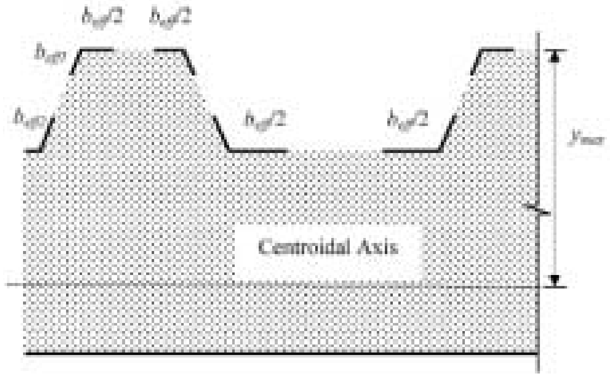


Figure 12 Typical Cross-Section used in the Effective Width Calculation

$$M_u = \frac{w_u L^2}{8} = \frac{f_y}{y_{\max}} I_{eff} \tag{23}$$

$$w_u = \frac{8 f_y I_{eff}}{y_{\max} L^2} \tag{24}$$

where w_u is the load per unit length, L is the span of the panel, y_{max} is the distance between the centroid and the topmost fibre of the profiled steel face and f_y is the yield stress of the steel. The load w_u was converted to a failure pressure p_u by using Equation 25 and then compared with the experimental failure pressure.

$$p_u = \frac{8 f_y I_{eff}}{y_{\max} B L^2} \tag{25}$$

where B is the overall panel width.

Table 3 presents the test results and their comparisons with the predictions from the design rules. As seen from this table, the results obtained from the new design rule are in very good agreement with the experimental results although they are slightly conservative. On the other hand, CIB[4] design rule always overestimates the panel strength in comparison with the experimental results. The mean value of the ratio of the failure pressures predicted by the new design rule and the experiments was found to be 0.94 and the corresponding coefficient of variation (COV) was 0.04. However, the mean value of the ratio of the failure pressures predicted from the current design rule[4] and the experiments was 1.08 and the corresponding COV was 0.04. This shows that the mean of the ratio of the failure pressures from CIB[4] and the experiments is consistently high and is greater than one. This implies that the CIB[4]design rule is not safe to use in the design of panels considered in this research.

Table 3
Comparison of Failure Pressure

Test No.	Panel Type	Experimental Failure Pressure (kPa)	Predicted Failure Pressure (kPa)			
			New Design Rule	Ratio New design/Expt.	CIB 2000	Ratio CIB 2000/ Expt.
1	A	7.65	7.49	0.98	8.58	1.12
2	A	5.17	4.62	0.90	5.30	1.03
3	A	3.72	3.33	0.90	3.81	1.02
4	B	9.93	9.21	0.93	10.60	1.07
5	B	7.05	6.86	0.97	7.90	1.12
6	B	5.84	5.69	0.97	6.55	1.12
Mean				0.94		1.08
Coefficient of Variation (COV)				0.04		0.04

Since the profiled steel faces used were made of 0.42 mm G550 steels, the design predictions from both CIB[4] and the new design rule were based on a reduced yield stress of $0.9f_y$. It is possible that the assumed reduced value of yield stress might be too conservative to use for sandwich panels, in which case, the predicted failure pressures from both the design rules should be considered as lower bound values. If the actual yield stress is used, design predictions based on both design rules would be higher than the values shown in Table 3. In this case, the predictions from the new design rule would be very close to experimental results and less conservative. But the predictions based on the CIB[4] design rule would be even more unsafe compared with the experimental results.

It should be noted that there are several factors that affect the experimental strength of sandwich panels. One such important factor is the bonding between the steel faces and the foam core. In the Australian method of manufacturing of Type B sandwich panels, attachment of the foam core with the steel face at the profiled ridge part is rather complicated. Without the use of a good manufacturing method and workmanship, it is difficult to ensure adequate bonding in that part of the panels. Inadequate bonding ultimately results in the reduction of panel strength. In the absence of such problems, the predictions from the new design rule are very close to the true strength of the profiled sandwich panels. Hence it is recommended that the new design rule is used in the design of profiled sandwich panels subjected to local buckling effects.

7 CONCLUSIONS

This paper has presented an overview of recent research into the behaviour of sandwich panels subject to local buckling effects. It included the details of an extensive series of experiments, finite element analyses and development of a new design rule. The study showed that the currently used conventional effective width approach is unconservative for the design of sandwich panels with slender plates and therefore a new improved design rule was developed for the design of fully profiled sandwich panels with any practical b/t ratios (< 600). A series of full-scale tests of six fully profiled sandwich panels was conducted to examine the accuracy of the new design rule. Test results indicated that the current design rule overestimated the strength of sandwich panels whereas the new improved design rule agreed well with the experimental results. Hence, the new design rule is recommended for the design of profiled sandwich panels to achieve safe design solutions.

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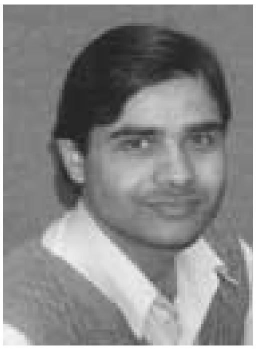

NARAYAN POKHAREL

Narayan Pokharel obtained his **Bachelor of Engineering** (B.E.) degree in civil engineering from Tribhuvan University, Nepal in 1994, **Master of Engineering** (M.E.) degree in structural engineering from Asian Institute of Technology (AIT), Bangkok, Thailand in 1998 and **PhD Degree** in structural engineering from Queensland University of Technology, Brisbane, Australia in 2004. After his bachelors degree, Narayan worked as a **government officer** in the Ministry of Housing and Physical planning, Kathmandu, Nepal for two years. He was a **research assistant** at AIT, Bangkok following the completion of his masters degree. He then moved to the National University of Singapore to undertake further research for about one and a half years before joining QUT in 2000.

MAHEN MAHENDRAN

Mahen Mahendran is a Professor in the School of Civil Engineering at Queensland University of Technology. He obtained his BScEng degree with a first class honours from the University of Sri Lanka in 1980 and his PhD from Monash University in 1985. He has since worked as an academic and applied researcher at various universities including James Cook University Cyclone Testing Station. His current research interests include behaviour and design of profiled steel cladding and sandwich panel systems under high wind forces, full scale behaviour of steel building systems and components, cyclone/storm-resistant building systems, buckling and collapse behaviour of thin-walled steel structures, and fire safety of steel buildings.