Appendix C :

Theoretical Analyses of Composite Panel

Structural Analyses Of Composite Foam Panels

Part I: Wind And Snow Load Conditions

By Raymond Ting, Ph.D., P.E.
Advanced Engineering Services

Introduction

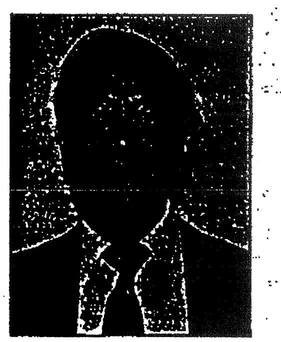
A composite foam panel is a building product consisting of a structural foam core sandwiched between two light gauge metal skins. The exterior metal skin is known as fascia and the interior metal skin is known as liner.

The popularity of composite foam panels in the construction of a building envelope can be attributed to the economy achieved by the dual functions of the foam core. The foam core is utilized to provide thermal insulation and to act as a shear transferring device between the metal skins to achieve the structural composite action.

Composite foam panels normally range in width from 24" to 42". The depth of the panel normally ranges from 1.5" to 12". The metal skins are profiled along the longitudinal edges to provide interlocking sealed panel joints. In most cases, the liner skin is essentially flat except at the side joint areas. The fascia skin can be essentially flat (flat panel) or profiled into various ribbed patterns (ribbed panel). For a ribbed panel, the shear force under load is shared by the foam core and the webs of the ribbed fascia skin. For a flat panel, the shear force under load can be assumed to be completely carried by the foam core. Therefore, a flat panel is in fact a particular condition of a generalized ribbed panel.

In order to serve as a useful reference for readers, this article is divided into two parts, each arranged in easy-to-follow sections. This first installment deals specifically with wind and snow load conditions while part two (appearing in the October 1993 issue of Metal Architecture) deals with thermal loads. Both parts are similarly arranged to include a Definition of Symbols and a Summary of Equations which are followed by Design Theories; Panel Properties; Example Analyses; and Derivations of Equations.

About The Author



Dr. Raymond Ting, PH.D, P.E., is the president of Advanced Engineering Services, a Pittsburgh-based firm specializing in light gauge metal building product research and engineering with an emphasis on composite building product applications. Dr. Ting has more than 20 years' experience in active building material research involving evaluation of product performance parameters and development of product design theory. He holds more than 30 patents in the United States and other countries in the area of light gauge metal building product or systems design. Prior to venturing into private practice, Dr. Ting spent 10 years in product research at H.H. Robertson Co. and performed consulting work in the area of composite panels for Aluma Shield Industries, Alumax Building Products, E.G. Smith Construction Products, Insulated Panel Systems, Bally Engineered Components, Molenco, Pond Hill Homes, HF Industries, Ryan Homes, Hesco of Saudi Arabia, and Yung Hsiang Special Steel in Taiwan. Ting will be a featured speaker at Metalcon International '93 and can be reached by writing to his company at 505A McKnight Park Drive, Pittsburgh, PA 15237, or calling (412) 367-7990.

Definitions Of Symbols

A_c = cross-sectional area of foam core (in^2/ft)
C_{dci} = composite midspan deflection coefficient of the ith span
C_{dni} = non-composite midspan deflection coefficient of the ith span
C_{mij} = composite moment coefficient at the ith support for the jth span
C_{nij} = non-composite moment coefficient at the ith support for the jth span
E = Young's modulus of metal skin (psi)
F_b = maximum liner stress (psi)
F_t = maximum fascia stress (psi)
F_{u1} = fascia buckling strength (psi)
F_{u2} = liner buckling strength (psi)
F_{uv} = core shear strength (psi)
F_v = maximum core shear stress (psi)
G_c = shear modulus of foam core (psi)
I_c = composite moment of inertia (in^4/ft)
I_n = non-composite moment of inertia of the fascia skin (in^4/ft)
k_i = ratio of composite bending stiffness to composite shear stiffness of the ith span
L_i = span length of the ith span (ft)
M_c = maximum composite moment (ft-#/ft)
M_{ci} = composite moment at the ith support (ft-#/ft)
M_{mi} = composite midspan moment of the ith span (ft-#/ft)
M_n = maximum non-composite moment (ft-#/ft)
M_{ni} = non-composite moment at the ith support (ft-#/ft)
M_{ri} = non-composite midspan moment of the ith span (ft-#/ft)
P_{ue} = connection strength at end support (#)
P_{ui} = connection strength at intermediate support (#)
R_{ci} = composite load distribution coefficient of the ith span
R_{ni} = non-composite load distribution coefficient of the ith span
S_b = composite liner section modulus (in^3/ft)
S_n = non-composite fascia section modulus (in^3/ft)
S_t = composite fascia section modulus (in^3/ft)
V_c = maximum shear force in core (#/ft)
W_i = design uniform load on the ith span (psf)
W_{ci} = composite uniform load on the ith span (psf)
W_{ni} = non-composite uniform load on the ith span (psf)
Y_{ci} = composite midspan deflection of the ith span (in)
Y_{ni} = non-composite midspan deflection of the ith span (in)

Summary of Equations

(1) $k_i = E I_c / (144 A_c G_c L_i^2)$
(2) $R_{ci} = 5 I_c / [5 I_c + 16 I_n (0.3125 + 3 k_i)]$
(3) $R_{ni} = 1 - R_{ci}$
(4) $W_{ci} = R_{ci} W_i$
(5) $W_{ni} = R_{ni} W_i$
(6) Three-Moment Equation applied to composite action:
$(6k_1-1)L_1 M_{c1} - [(2+6k_1)L_1+(2+6k_2)L_2] M_{c2} + (6k_2-1) L_2 M_{c3} = (W_{c1} L_1^3 + W_{c2} L_2^3) / 4$
(7) Three-Moment Equation applied to non-composite action:
$-L_1 M_{n1} - 2(L_1 + L_2) M_{n2} - L_2 M_{n3} = (W_{n1} L_1^3 + W_{n2} L_2^3)/4$
(8) $C_{mij} = M_{ci} / (W_{cj} L_j^2)$
(9) $C_{nij} = M_{ni} / (W_{nj} L_j^2)$
(10) $C_{dci} = R_{ci} [0.3125 + 3 k_i + 1.5 (C_{mi} + C_{mi+1})] / 24$
(11) $Y_{ci} = 1728 C_{dci} W_i L_i^4 / (E I_c)$
(12) $C_{dni} = R_{ni} [0.3125 + 1.5 (C_{ni} + C_{ni+1})] / 24$
(13) $Y_{ni} = 1728 C_{dni} W_i L_i^4 / (E I_n)$
(14) $Y_{ci} = Y_{ni}$
(15) $R_{ci} + R_{ni} = 1$
(16) $M_{mi} = R_{ci} W_i L_i^2 / 8 + (M_{ci} + M_{ci}+1) / 2$
(17) $M_{ri} = R_{ni} W_i L_i^2 / 8 + (M_{ni} + M_{ni}+1) / 2$
(18) $F_b = 12 M_c / S_b$
(19) $F_t = 12 (M_c / S_t + M_n / S_n)$
(20) $F_v = V_c / A_c$

Steps Included In The Procedures Of Analysis

1. Calculate the values of K_i and the initial values of R_{ci} and R_{ni} using Equations 1 to 3.
2. Use Equations 4 to 7 to solve the intermediate support moments M_{ci} and M_{ni}.
3. Use Equations 8 to 13 to express Y_{ci} as a function of R_{ci} and Y_{ni} as a function of R_{ni}.
4. Substitute the results of Step 3 into Equation 14 and solve Equations 14 and 15 simultaneously for R_{ci} and R_{ni}.
5. Repeat from Step 2 using the new values of R_{ci} and R_{ni} obtained in Step 4 until the values of R_{ci} and R_{ni} converge to the desired accuracy.
6. Use Equation 11 and Equations 16 to 20 to complete the structural analyses.

Note: For a flat panel product, the values of R_{ci} are equal to 1.0 and the values of M_{ni} are equal to zero. The applicable equations are 1, 4, 6, 8, 10, 11, 16, and 18 to 20. There is no repeating procedure involved.

Design Theory

The determination of the allowable load includes two major considerations, namely, stiffness and strength. A maximum midspan deflection of span/180 is generally acceptable in the industry for the composite panel design. The generally acceptable safety factors for the composite panel design are tabulated in Table 1 below.

	Design Safety Factor	
ITEM	SNOW LOAD	WIND LOAD
Metal Skin Buckling	2.5	1.875
Connection Strength	N.A.	1.875
Core Shear Strength	3.0	3.000

Table 1.

Panel Properties

The panel properties can be classified into two categories, namely, theoretical properties and empirical properties. The theoretical properties are obtained by calculations in accordance with the linear elastic theory including the cross-sectional area of foam core, the composite moment of inertia, the non-composite moment of inertia of the fascia skin, the composite section modulus and the non-composite fascia section modulus. The empirical properties are derived from the results of structural load tests including the shear modulus of the foam core, the Young's modulus of the metal skin, the buckling strength of the metal skins and the shear strength of the foam core. Due to the fact the foam core is not perfectly uniform or isotropic, the empirical composite properties derived from small scale load tests are not reliable. Therefore, full scale load tests should be exclusively used for evaluating the empirical composite properties.

The foam core exhibits a significant degree of strain creep under sustained long term loading. Therefore, long term load tests should be used to determine the shear creep factor of the foam core for snow load design. Depending on the manufacturing process, the shear modulus of the foam core is often affected by the panel depth. With adequate test data, least square method should be employed to obtain the empirical properties. With expanded test data, the theoretical properties can be modified by empirical effective coefficients to improve the engineering precision. Any published design data should be critically examined in the contents of test data base and design theory.

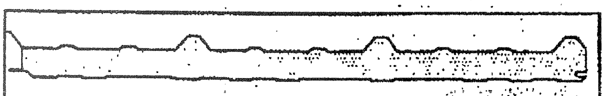


Figure 1: Typical Profile Of Ribbed Panel

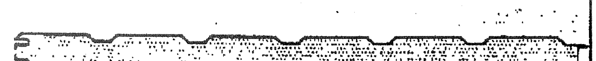


Figure 2: Typical Profile Of Essentially Flat Panel

Example Analysis No. 1
Ribbed Panel (as shown in Figure 1

[Given]

Theoretical Properties:
A_c = 26.03 in²/ft
I_c = 0.656 in⁴/ft
S_b = 0.529 in³/ft
S_t = 0.609 in³/ft
I_n = 0.0499 in⁴/ft
S_n = 0.0536 in³/ft

Empirical Properties:
G_c = 301.73 psi
E = 29000000 psi
F_{u1} = 40000 psi
F_{u2} = 23500 psi
F_{uv} = 24.6 psi

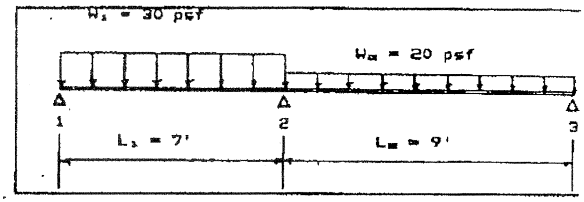


Figure 3: Span and loading conditions:

[Solution]

Item

$k_1 = 29000000 \times 0.656/(144 \times 26.03 \times 301.73 \times 7^2) = 0.3433$ (Eq.
$k_2 = 29000000 \times 0.656/(144 \times 26.03 \times 301.73 \times 9^2) = 0.2077$ (Eq.

First Repeating Cycle:

$R_{c1} = 5 \times 0.656/[5 \times 0.656 + 16 \times 0.0499(0.3125 + 3 \times 0.3433)] = 0.754$ (Eq.
$R_{n1} = 1 - 0.754 = 0.246$ (Eq.
$R_{c2} = 5 \times 0.656/[5 \times 0.656 + 16 \times 0.0499(0.3125 + 3 \times 0.2077)] = 0.815$ (Eq.
$R_{n2} = 1 - 0.815 = 0.185$ (Eq.
$W_{c1} = 0.754 \times 30 = 22.62$ psf (Eq.
$W_{n1} = 0.246 \times 30 = 7.37$ psf (Eq.
$W_{c2} = 0.815 \times 20 = 16.3$ psf (Eq
$W_{n2} = 0.185 \times 20 = 3.7$ psf (Eq
$-[(2 + 6 \times 0.3433) \times 7 + (2 + 6 \times 0.2077) \times 9] M_{c2} = (22.62 \times 7^3 + 16.3 \times 9^3)/4$ (Eq
i.e. $-57.63 M_{c2} = 4910.34$
$Mc2 = -4910.34/57.63 = -85.2$ ft-#/ft $-2(7 + 9) M_{n2} = 7.38 \times 7^3 + 3.7 \times 9^3)/4$ (Eq
i.e. $-32 M_{n2} = 1307.16$
$M_{n2} = -1307.16/32 = -40.8$ ft-#/ft

Span No. 1:

$C_{m21} = -85.2/(22.62 \times 7^2) = -0.0769$ (Eq.
$C_{n21} = -40.8/(7.38 \times 7^2) = -0.1128$ (Eq
$C_{dc1} = Rc1\ (0.3125 + 3 \times 0.3433 - 1.5 \times 0.0769)/24 = 0.0511 R_{c1}$ (Eq.
$Y_{c1} = 1728(0.0511 R_{c1}) \times 30 \times 7^4/(29000000 \times 0.656) = 0.3345 R_{c1}$ (Eq.
$C_{dn1} = R_{n1}\ (0.3125 - 1.5 \times 0.1128)/24 = 0.00597 R_{n1}$ (Eq.
$Y_{n1} = 1728(0.00597 R_{n1}) \times 30 \times 7^4/(29000000 \times 0.0499) = 0.5136 R_{n1}$ (Eq.
$0.3345 R_{c1} = 0.5136 R_{n1}$ (Eq.
$R_{c1} + R_{n1} = 1$ (Eq.1

Solving the above two equations simultaneously yields

$R_{c1} = 0.6056$
$R_{n1} = 0.3944$

Span No. 2:

$C_{m22} = -85.2/(16.3 \times 9^2) = -0.0645$ (Eq.
$C_{n22} = -40.8/(3.7 \times 9^2) = -0.1361$ (Eq
$C_{dc2} = R_{c2}(0.3125 + 3 \times 0.2077 - 1.5 \times 0.0645)/24 = 0.03495 R_{c2}$ (Eq.
$Y_{c2} = 1728(0.03495 R_{c2}) \times 20 \times 9^4/(29000000 \times 0.656) = 0.4166 R_{c2}$ (Eq.
$C_{dn2} = R_{n2}(0.3125 - 1.5 \times 0.1361)/24 = 0.004515 R_{n2}$ (Eq.
$Y_{n2} = 1728(0.004515 R_{n2}) \times 20 \times 9^4/(29000000 \times 0.0499) = 0.7074 R_{n2}$ (Eq.
$0.4166 R_{c2} = 0.7074 R_{n2}$ (Eq.
$R_{c2} + R_{n2} = 1$ (Eq.

Solving the above two equations simultaneously yields

$R_{c2} = 0.6294$
$R_{n2} = 0.3706$

Second Repeating Cycle:

$W_{c1} = 0.6056 \times 30 = 18.17$ psf (Eq
$W_{n1} = 0.3944 \times 30 = 11.83$ psf (Eq
$W_{c2} = 0.6294 \times 20 = 12.59$ psf (Eq
$W_{n2} = 0.3706 \times 20 = 7.41$ psf (Eq

Continued on page

$-[(2 + 6x0.3433) x 7 + (2 + 6x0.2077) x 9] M_{c2} = (18.17 x 7^3 + 12.59 x 9^3)/4$ (Eq. 6)

i.e. $-57.63 M_{c2} = 3852.61$

$M_{c2} = -3852.61/57.63 = -66.85$ ft-#/ft $-2(7 + 9) M_{n2} = (11.83 x 7^3 + 7.41 x 9^3)/4$ (Eq. 7)

i.e. $-32 M_{n2} = 2364.9$

$M_{n2} = -2364.9/32 = -73.9$ ft-#/ft

Span No. 1:

$C_{m21} = -66.85/(18.17 x 7^2) = -0.0751$ (Eq. 8)

$C_{n21} = -73.9/(11.83 x 7^2) = -0.1275$ (Eq. 9)

$C_{dc1} = R_{c1} (0.3125+3x0.3433-1.5x0.0751)/24 = 0.05124 R_{c1}$ (Eq. 10)

$Y_{c1} = 1728(0.05124 R_{c1})x30x7^4/(29000000x0.656) = 0.3352 R_{c1}$ (Eq. 11)

$C_{dn1} = R_{n1} (0.3125-1.5x0.1275)/24 = 0.00505 R_{n1}$ (Eq. 12)

$Y_{n1} = 1728(0.00505 R_{n1})x30x7^4/(29000000x0.0499) = 0.4345 R_{n1}$ (Eq. 13)

$0.3352 R_{c1} = 0.4345 R_{n1}$ (Eq. 14)

$R_{c1} + R_{n1} = 1$ (Eq. 15)

Solving the above two equations simultaneously yields

$R_{c1} = 0.5645$

$R_{n1} = 0.4355$

Span No. 2:

$C_{m22} = -66.85/(12.59 x 9^2) = -0.0656$ (Eq. 8)

$C_{n22} = -73.9/(7.41 x 9^2) = -0.1231$ (Eq. 9)

$C_{dc2} = R_{c2} (0.3125+3x0.2077-1.5x0.0656)/24 = 0.03488 R_{c2}$ (Eq. 10)

$Y_{c2} = 1728 (0.03488 R_{c2})x20x9^4/(29000000x0.656) = 0.4158 R_{c2}$ (Eq. 11)

$C_{dn2} = R_{n2}(0.3125-1.5x0.1231)/24 = 0.005327 R_{n2}$ (Eq. 12)

$Y_{n2} = 1728(0.005327 R_{n2})x20x9^4/(29000000x0.0499) = 0.8347 R_{n2}$ (Eq. 13)

$0.4158 R_{c2} = 0.8347 R_{n2}$ (Eq. 14)

$R_{c2} + R_{n2} = 1$ (Eq. 15)

Solving the above two equations simultaneously yields

$R_{c2} = 0.6675$

$R_{n2} = 0.3325$

For demonstration purposes only, this cycle is considered to be final. More cycles can be performed to achieve more accurate results.

$W_{c1} = 0.5645 x 30 = 16.94$ psf (Eq. 4)

$W_{n1} = 0.4355 x 30 = 13.06$ psf (Eq. 5)

$W_{c2} = 0.6675 x 20 = 13.35$ psf (Eq. 4)

$W_{n2} = 0.3325 x 20 = 6.65$ psf (Eq. 5)

$-[(2 + 6x0.3433)x7 + (2 + 6x0.2077)x9] M_{c2} = (16.94 x 7^3 + 13.35 x 9^3)/4$ (Eq. 6)

i.e. $-57.63 M_{c2} = 3885.64$

$M_{c2} = -3885.64/57.63 = -67.42$ ft-#/ft $-2(7 + 9) M_{n2} = (13.06 x 7^3 + 6.65 x 9^3)/4$ (Eq. 7)

i.e. $-32 M_{n2} = 2331.85$

$M_{n2} = -2331.85/32 = -72.87$ ft-#/ft

Span No. 1:

$C_{m21} = -67.42/(16.94 x 7^2) = -0.08122$ (Eq. 8)

$C_{n21} = -72.89/(13.06 x 7^2) = -0.1139$ (Eq. 9)

$C_{dc1} = 0.5645(0.3125+3x0.3433-1.5x0.08122)/24 = 0.02871$ (Eq. 10)

$Y_{c1} = 1728x0.02871x30x7^4/(29000000x0.656) = 0.1822"$ (Eq. 11)

$C_{dn1} = 0.4355(0.3125-1.5x0.1139)/24 = 0.00257$ (Eq. 12)

$Y_{n1} = 1728x0.0257x30x7^4/(29000000x0.0499) = 0.2211"$ (Eq. 13)

$Y_1 = (0.1822 + 0.2211)/2 = 0.2017" = L_1/416$ O.K.

Span No. 2:

$C_{m22} = -67.42/(13.35 x 9^2) = -0.06235$ (Eq. 8)

$C_{n22} = -72.89/(13.06 x 9^2) = -0.1353$ (Eq. 9)

$C_{dc2} = 0.6675(0.3125+3x0.2077-1.5x0.06235)/24 = 0.02342$ (Eq. 10)

$Y_{c2} = 1728x0.02342x20x9^4/(29000000x0.656) = 0.2791"$ (Eq. 11)

$C_{dn2} = 0.3325(0.3125-1.5x0.1353)/24 = 0.001518$ (Eq. 12)

$Y_{n2} = 1728x0.001518x20x9^4/(29000000x0.0499) = 0.2378"$ (Eq. 13)

$Y_2 = (0.2791 + 0.2378)/2 = 0.2585" = L_2/418$ O.K.

Midspan Bending Moments

$M_{m1} = 0.5645 x 30 x 72/8 - 67.42/2 = 70.02$ ft-#/ft (Eq. 16)

$M_{m2} = 0.6675 x 20 x 92/8 - 67.42/2 = 101.46$ ft-#/ft (Eq. 16)

$M_{r1} = 0.4355 x 30 x 72/8 - 72.87/2 = 43.59$ ft-#/ft (Eq. 17)

$M_{r2} = 0.3325 x 20 x 92/8 - 72.87/2 = 30.90$ ft-#/ft (Eq. 17)

Maximum Liner Bending Stress

$F_b = 12 x 101.46/0.529 = 2302$ psi (Eq. 18)

$S.F. = F_{u2}/F_b = 23500/2302 = 10.2$ O.K.

Maximum Fascia Bending Stress

At Support: $F_{ts} = 12(67.42/0.609 + 72.87/0.0536) = 17643$ psi (Eq. 19)

At Midspan: $F_{tm} = 12(101.46/0.609 + 30.9/0.0536) = 8917$ psi (Eq. 19)

$F_t = F_{ts} = 17643$ psi

$S.F. = F_{u1}/F_t = 40000/17643 = 2.267$ O.K.

Maximum Core Shear Stress

$V_c = 0.5645 x 30 x 7/2 + 67.42/7 = 68.9$ #/ft

$F_v = 68.9/26.03 = 2.65$ psi (Eq. 20)

$S.F. = F_{uv}/F_v = 24.6/2.65 = 9.28$ O.K.

Example Analysis No. 2: Essentially Flat Panel (as shown in Figure 2)

[Given]

Theoretical Properties:	Empirical Properties
$A_c = 22.27$ in²/ft	$G_c = 557$ psi
$I_c = 0.434$ in⁴/ft	$E = 29000000$ psi
$S_b = 0.440$ in³/ft	$F_{u1} = 24000$ psi
$S_t = 0.428$ in³/ft	$F_{u2} = 22000$ psi
	$F_{uv} = 30$ psi

Span and loading conditions are shown on Figure 3.

[Solution]

Item	Eq.
$k_1 = 29000000x0.434/(144x22.27x557x7^2) = 0.1438$	(Eq. 1)
$k_2 = 29000000x0.434/(144x22.27x557x9^2) = 0.087$	(Eq. 1)
For essentially flat panel, $R_{c1} = R_{c2} = 1.0$	
$-[(2 + 6x0.1438)x7 + (2 + 6x0.087)x9] M_{c2} = (30 x 7^3 + 20 x 9^3)/4$	(Eq. 6)
i.e. $-42.74 M_{c2} = 6217.5$	
$M_{c2} = -6217.5/42.74 = -145.48$ ft-#/ft	
Span No. 1:	
$C_{m21} = -145.48/(30 x 7^2) = -0.099$	(Eq. 8)
$C_{dc1} = (0.3125 + 3x0.1438 - 1.5x0.099)/24 = 0.02481$	(Eq. 10)
$Y_1 = Y_{c1} = 1728x0.02481x30x7^4/(29000000x0.434)$	(Eq. 11)
$= 0.245" = L_1/344$ O.K.	
Span No. 2:	
$C_{m22} = -145.48/(20 x 9^2) = -0.0898$	(Eq. 8)
$C_{dc2} = (0.3125 + 3x0.087 - 1.5x0.0898)/24 = 0.01828$	(Eq. 10)
$Y_2 = Y_{c2} = 1728x0.01828x20x9^4/(29000000x0.434)$	(Eq. 11)
$= 0.329" = L_2/328$ O.K.	
Midspan Bending Moments	
$M_{m1} = 30 x 7^2/8 - 145.48/2 = 111.01$ ft-#/ft	(Eq. 16)
$M_{m2} = 20 x 9^2/8 - 145.48/2 = 129.76$ ft-#/ft	(Eq. 16)
Comparing the absolute values of M_{c2}, M_{m1}, M_{m2} *yields* $M_c = 145.48$ *ft-#/ft*	
Maximum Bending Stresses	
$F_b = 12 x 145.48/0.44 = 3968$ psi	(Eq. 18)
$S.F. = F_{u2}/F_b = 22000/3968 = 5.54$ O.K.	
$F_t = 12 x 145.48/0.428 = 4079$ psi	(Eq. 19)
$S.F. = F_{u1}/F_t = 24000/4079 = 5.88$ O.K.	
Maximum Core Shear Stress	
$V_c = 30 x 7/2 + 145.48/7 = 125.78$ #/ft	
$F_v = 125.78/22.27 = 5.65$ psi	(Eq. 20)
$S.F. = F_{uv}/F_v = 30/5.65 = 5.31$ O.K.	

Derivations of Equations

In a ribbed composite foam panel, the total shear force acting on a cross-section of the panel is shared by the webs of the ribbed skin and the foam core. The shear resisted by the webs of the ribbed skin produces cross-sectional rotation about the neutral axis of the non-composite ribbed skin (i.e. non-composite action). The shear resisted by the foam core produces cross-sectional rotation about the composite neutral axis (i.e. composite action). Under the effect of composite action, the shear deflection is a significant part of the total deflection. Under the effect of non-composite action, there is negligible shear deflection. The compatibility of deflection requires that the deflection due to non-composite action must be equal to that due to composite action at any point along the entire length of the panel. Using this compatibility requirement, the loads resisted by the composite action and the non-composite action can be determined (i.e. total load = non-composite load + composite load). Once the non-composite load and the composite load have been determined, the structure can be fully analyzed by superimposing the effects of the two component loads (e.g. total stress = non-composite stress + composite stress). However, for a given loading condition, the deflected shape due to composite action which includes both bending and shear deflections is different from the deflected shape due to non-composite action which does not have shear deflection. To force the deflected shape into the same mode due to the compatibility requirement require that the shape of the composite loading function be different from the non-composite loading function. Rational analysis leads to very complex loading functions which become impractical for design purposes. To simplify the analysis, it is assumed that for a uniform loading condition, the non-composite and the composite loading functions remain uniform and the compatible deflection is only required at each midspan. Equation (2) is derived from this assumption for a simple span structure without the end moments. For a continuous span structure, there are two methods commonly employed by the practicing engineers. The first method uses the intermediate support reactions as the redundant forces with the zero support deflections as the compatibility requirements. The second method uses the intermediate support moments as the redundant forces with compatible cross-sectional rotations at supports as the compatibility requirements. The formulations of the first method are complicated in the case of unequal span condition or stepwise uniform loads. It is much desirable to use the second method which leads to the so called Three-Moment Equation. Equation (6) is the Three-Moment Equation derived from the second method for the composite structure under the effect of the composite load. Equation (7) is the Three-Moment Equation derived from the second method for the non-composite structure under the effect of the non-composite load. Due to the fact that the composite and the non-composite load distribution coefficients and the redundant moments are inter-related, converging trial-and-error procedure is most practical in solving the indeterminate continuous span structure. Equations (8) and (10) are derived from the simple beam structure of each span with the known redundant end moments. Another engineering simplification is to consider the midspan deflection as the maximum deflection. For an essentially flat panel, the non-composite action can be ignored and the equations simplified accordingly. The equations simplified for flat panels are applicable for the analyses of paper or aluminum honeycomb panels.

Part II of Dr. Ting's article will appear in the October 199[illegible] issue of *Metal Architecture*. Dr. Ting will review much of the information found in this article as he and Tom Shingler of AEP-Span team up at Metalcon International '9[illegible] to host a two-part seminar dealing with composite foam panel analysis. For more information about the trade show or the seminar, please see the Official Metalcon Show Guide, bound into this issue.

Novatek Acquires Engineering / Fabricating Company

Novatek International, a developer, manufacturer and supplier of pre-engineered steel frame and concrete panel building components, has announced the acquisition of TruSteel Engineering Co. Inc., a light steel engineering and steel fabricating firm.

In a related development, Novatek has acquired the exclusive marketing rights from an affiliate of TruSteel for a newly developed computer software program that will facilitate the design and accuracy of cost estimates for steel trusses.

The announcement was made jointly by Novatek President Frank Cooney and TruSteel owners and founders Thomas Kelaher and Ian Yap.

TruSteel will continue to operate as in the past, providing construction and engineering design services for light steel commercial and residential construction. The company will move its operation from Deerfield Beach to Novatek's corporate and manufacturing facility in Boynton Beach.

"TruSteel is perhaps the most advanced and creative engineering firm serving the burgeoning light steel construction industry," said Cooney. "The firm has established a distinguished reputation for excellence and we anticipate the opening of TruSteel branch offices throughout the United States to meet the rapidly increasing demand for high quality light steel engineering services.