nidaplast®
HONEYCOMBS

Sandwich Panel Design

1 – GENERAL INFORMATION

Sandwich panels are often used instead of monolithic panels to provide adequate rigidity with less weight. The stressed faces, one in tension and the other in compression interact with the core through the joints by providing a large bending stiffness. The core of the polypropylene honeycomb transmits the shearing strains between the faces, and by increasing the cores thickness, the panels total rigidity is increased to the required amount.

Several parameters are involved in the design of a sandwich panel:

- Consideration of deflection limits, design deflections or in-service deflections.
- The resistance to rupture of the joints and faces (component stresses), the shear stiffness and strength of the core, the stiffness of the faces (sizing the components).
- Is the panel statically or dynamically stressed? Should fatigue be considered?

The first two points are straightforward to determine by calculation or test. The last point is more difficult to consider by calculation only. If dynamic or fatigue properties are needed, physical testing in simulated or actual conditions will provide the best indication of performance.

2 – CALCULATIONS

The deflection behaviour and breaking strength (rupture) are two independent properties, which do not necessarily correlate. One can have large rigidity (low deflections) with low rupture strength, or on the contrary have a low rigidity panel (large deflections) with great resistance to rupture (high strength).

The polypropylene honeycomb NIDAPLAST, has a low shear modulus and must be considered differently to other core materials. With equal core thickness to a typical foam core and having the same faces, the Nidaplast panel may be slightly less rigid, but will have similar flexural strength.

2.1. DEFLECTIONS

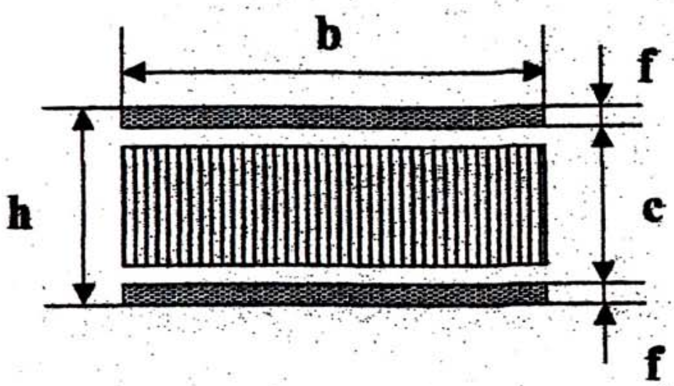
In general it is the design deflection limits which dimension the panel (ie panel stiffness), then the core shear strength is checked, then the stress state of the faces is checked.

Rupture is expected to be reached only for deflections much larger than the working load limits allow. A sandwich panels total deflection, when used as a beam consists of the sum of two components,

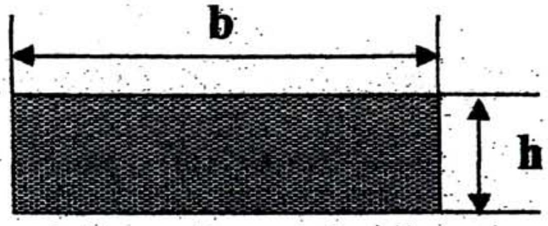
1) Its fibre bending component and 2) its shear bending component. The fibre bending stiffness is provided by the faces, and the shear stiffness is provided by the core. In engineering beams the "shear bending" component is only a few percent of the total deflection and is normally neglected. Core materials are not stiff, which means the shear deflections are significant in a sandwich panel. This is why they are considered in the following analysis.

Consider the resultant deflections, of a sandwich panel working as a beam. This can be approached in a similar way to that of a monolithic beam, as shown below:

Sandwich Panel



Monolithic Panel



Ef: Modulus of faces
Gc: Shear Modulus of the core

E: Bulk Modulus of the panel

P: total load; **L**: span between supports; **I**: second moment of inertia

The total deflection Y is given by:

$$Y = y_1 + y_2$$

$$Y = y_1 \text{ (neglect shear deflection } y_2\text{)}$$

- where y_1 is the "fibre bending deflection" component,

$$y_1 = K_f PL^3/E_f I$$

$$y_1 = K_f PL^3/EI$$

The fibre deflection, y1 is proportional to the cube of the span, and is in general the dominating factor, especially as the span gets larger. In a given configuration of load and span, the way to decrease the deflection is to increase the rigidity. **R** = $E_f I$

To increase the rigidity, we can;

a) Increase the modulus, Ef of the faces. By changing the material or the laminate.
b) Increase the inertia **I** = $bh^3/12$, by increasing the core thickness (h being cubed, therefore being dominant). This is the most economic solution from which the sandwich structure is most widely designed and understood.

- y_2 is the "shear deflection" component,

$$y_2 = K_s \, PL/b\,(c+f)Gc$$

The shear deflection y_2, is mainly resultant from Gc being inversely proportional to y2. This means the lower the Gc the larger the deflection.

Note. *Appendix 1* presents the coefficients K_s and Kf for various possible load cases.

2.2 DESIGN LIMITS

The compression and tensile Fibre stresses (Fσf) in the faces are given by the following formula:

$$F\sigma f = 2M/(b.f\,(c+f))$$

and the Shear stress (Sσc) in the core by the formula:

$$S\sigma c = N/(b.(c+f))$$

Where M is the bending moment and N is the shearing force.

It is necessary to check that the calculated working stresses are lower than those of the design max stresses. The maximum working/design stresses are determined by using a suitable service factor on the known or predicted rupture stresses.

3 – SUMMARY

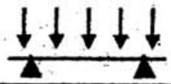
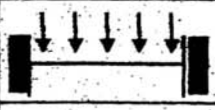
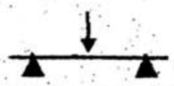
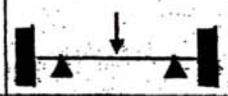
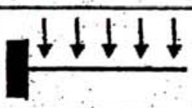
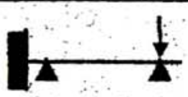
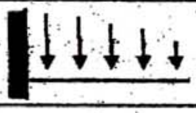
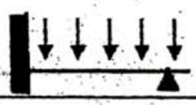
The above simplified calculations examine the deflections and the stress limits under the effect of a static load. A static flexure test makes it possible to check if the calculations and the manufacture of the panel are accurate. These tests and calculations do not indicate the fatigue performance or panel deflections under dynamic stress conditions.

As discussed previously, the NIDAPLAST honeycombs have a low shear modulus compared to their shearing strength, ie it is very tough. This toughness provides good damage tolerance properties. A NIDAPLAST sandwich will deform and distort much more than a foamed cored panel prior to rupture.

These calculations are preliminary only and do not represent a thorough analysis. Other failure modes of a panel are possible and should be checked for. Eg Buckling, Shear crimping, Wrinkling, Core tensile/comp failure and Dimpling.

APPENDIX 1

The coefficients Kf & Ks, the bending moment M and the shearing force N appear below, in relation to various load cases.

Loadcases			N	M	Kf	Ks
Simply Supported Uniform Load	1		$\frac{P}{2}$	$\frac{PL}{8}$	$\frac{5}{384}$	$\frac{1}{8}$
Fixed ends Uniform Load	2		$\frac{P}{2}$	$\frac{PL}{12}$	$\frac{1}{384}$	$\frac{1}{8}$
Simply Supported Central Load	3		$\frac{P}{2}$	$\frac{PL}{4}$	$\frac{1}{48}$	$\frac{1}{4}$
Fixed Ends Central Load	4		$\frac{P}{2}$	$\frac{PL}{8}$	$\frac{1}{192}$	$\frac{1}{4}$
Fixed Ends Uniform Load	5		P	$\frac{PL}{2}$	$\frac{1}{8}$	$\frac{1}{2}$
Fixed Ends Point Load	6		P	PL	$\frac{1}{3}$	1
Fixed Ends Decreasing Load	7		P	$\frac{PL}{3}$	$\frac{1}{15}$	$\frac{1}{3}$
Semi Fixed Uniform Load	8		$\frac{5P}{8}$	$\frac{PL}{8}$	$\frac{1}{185}$	$\frac{1}{14,2}$

Footnote 1:
The resistance in flexure of a panel is directly related to the adherence between the skins and the core. It is therefore necessary to carry out physical testing to characterise and qualify the joint performance prior to use.

Footnote 2:
The panel deflection in flexure is related to two qualities, skin stiffness and core stiffness. It is very important to carry out the flexure test with a span close to the real span of the sandwich panel. For bending tests of panels, a distance between centres greater than 40 times the thickness is desirable. For spans less than this local shear effects may become significant and a more sophisticated approach to the analysis will be needed.

Example No1

Consider a panel being used as a beam in the type 3 configuration. Ie Centrally loaded with a 100kg mass. The Nidaplast is 20mm thick and 500mm wide, the faces are 2mm thick chopped stand mat. How far does the beam deflect in the middle? Will it rupture? Will it be reasonably safe at a span of 2m?

Loadcase data	P = 100kgf = 981N, K_f = 1/48, K_s = 0.25, N = 491N, M =490500Nmm, from Appendix I.
Material data	E_f = 8200MPa, nominal CSM specification G_c = 8MPa Nidaplast specification
Panel data	L = 2000mm, b=500mm, h=24mm, c=20mm, f=2mm

Strength Data

For "reasonable" safety use a service factor of 2.

UTS of CSM = 100MPa, ie use a max working stress of 100/2 = 50MPa.

Shear strength of core = 0.5MPa, use a max working stress of 0.5/2 = 0.25MPa.

Equations

1) Y = Kf.PL^3/Ef.I + Ks.PL/b(h+c)Gc central deflection.

2) Fσf = 2M/b.f(c+f) face stress 3) Sσc = N/b(c+f) shear stress

4) I = bh^3/12 Inertia

Calculate deflection...

Eqn 1, Y = Kf.PL^3/Ef.I + Ks.PL/b(h+c)Gc need to know If, Inertia of the faces...

Use Eqn 4, I = bh^3/12, ie (500x24^3/12 – 500x20^3/12) = 242667mm^4

Now Y = 1/48x981x2000^3/(8200x242667) + 0.25x981x2000/(500(20+2)8)

Y = 82mm fibre deflection + 5.6mm shear deflection

In a foam core the shear deflection would be about 1.5mm

Y = 88mm, it is up to the designer or code to determine if this is acceptable. With a typical foam core it would be about 84mm. Ie about a 5% difference.

Checking stresses...

Eqn 2 - Fσf = 2M/b.f(c+f) face stresses, Fσf = 2x490500/(500x2(20+2))

Fσf = 45Mpa < 50MPa ie is satisfactory.

Eqn 3 - Sσc = N/b(c+f) core shear stresses, Sσc = 491/500(20+2)

Sσc = 0.045MPa < 0.25MPa ie is satisfactory

Conclusion – The panel will not rupture and is stressed low enough to be reliable in this loadcase. The deflection limits have to be specified to decide if the calculated deflection is satisfactory. This is determined by its application, or from a suitable code of design.

Indicatives tables of Load for a bending of 1/200 of the unsupported length ref 6/97
sandwichs NIDAPLAST 8 , 2 mm GRP skin, E modulus 10000 MPa

e = Nidaplast thickness L = unsupported length [shaded] not valable too high shear

1) simple support, central loading

portée L cm	50	100	150	200	300
e mm	total load daN per m of width				
5	34	11	5	3	1
10	81	30	14	8	4
15	138	55	28	16	8
20	201	88	45	27	12
23	240	109	57	34	16
28	309	149	80	49	23
40	481	260	147	91	44
50	630	365	214	136	66
60	782	478	290	187	93
70	936	598	374	245	123
80	1 090	724	464	309	158
90	1 246	854	560	378	196

2) embed support, central loading (shaded: 50 cm column all rows; 100 cm column rows 50–90)

portée L cm	50	100	150	200	300
5	104	40	20	11	5
10	227	102	53	32	15
15	366	184	101	62	29
20	511	280	160	100	48
23	600	343	200	127	62
28	752	455	275	177	87
40	1 122	750	483	322	165
50	1 434	1 014	682	467	246
60	1 749	1 290	898	630	340
70	2 065	1 575	1 129	808	446
80	2 382	1 866	1 371	999	563
90	2 700	2 161	1 622	1 202	691

3) simple support, reparted loading (shaded: 50 cm column rows 28–90)

portée L cm	50	100	150	200	300
e mm	Load daN/m2				
5	115	18	6	2	1
10	283	49	16	7	2
15	490	92	30	13	4
20	725	148	49	22	7
23	874	185	63	28	9
28	1 135	255	89	40	12
40	1 798	450	164	75	24
50	2 376	640	241	112	36
60	2 970	846	329	156	51
70	3 575	1 067	427	205	67
80	4 184	1 300	533	259	86
90	4 802	1 545	647	319	107

4) embed support, reparted loading (shaded: 50 cm column all rows; 100 cm column rows 20–90; 150 cm column rows 60–90)

portée L cm	50	100	150	200	300
5	284	68	24	11	3
10	572	162	63	30	10
15	875	276	114	56	19
20	1 185	401	174	88	30
23	1 372	431	214	109	38
28	1 686	618	285	149	53
40	2 446	963	474	260	98
50	3 062	1 261	645	365	143
60	3 719	1 564	825	478	194
70	4 356	1 872	1 011	599	249
80	4 995	2 181	1 201	724	310
90	5 636	2 493	1 395	854	373