The design and analysis of buildings with light cladding

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Objekttyp: **Article**

Zeitschrift: **IABSE publications = Mémoires AIPC = IVBH Abhandlungen**

Band (Jahr): **28 (1968)**

PDF erstellt am: **18.09.2024**

Persistenter Link: https://doi.org/10.5169/seals-22178

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The Design and Analysis of Buildings with Light Cladding

Projection et dimensionnement de constructions à revêtements extra-légers

Entwurf und Berechnung von Gebäuden mit leichter Verkleidung

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Introduction

It has long been recognised that light cladding makes a contribution to the stiffness and strength of steel framed buildings but the effect has not been taken into account in design. This is probably because the cladding has been regarded as an uncertain element for structural purposes; it can be easily removed for maintenance and the methods of attachment have been variable and uncertain. In addition, steel sheets may corrode and the properties of other types of cladding may deteriorate with the passage of time. Also, no method has been available for estimating, even approximately, what the likely stiffening effect of the sheeting would be. Consequently, designers have tended to regard the effect as a "bonus" in reducing frame stresses and deflexions to values somewhat below the calculated values. In many cases, the only conscious use of the sheeting in design has been to allow the sheeting to provide lateral support to the purlins, but this must be regarded as a purely local effect which does not affect the overall behaviour of the building.

In recent years, however, a much more positive role for sheeting and decking has been adopted in the United States [1—4]. Welded floor decking has been used to provide resistance to wind and seismic forces, and light steel decking has been used for the shear diaphragms in folded plate roofs. In both these cases, design information has been based on the results of full scale tests.

In the design of industrial shed-type buildings, it has been shown [5] that the contribution of the sheeting to the frame stiffness and strength can be calculated provided that the shear behaviour of a panel of sheeting is known. In this connexion a panel is regarded as being that area of sheeting, complete with all attachments, between two adjacent rafters and between the extreme purlins. Originally, full scale tests were carried out to determine the shear behaviour of complete panels, though it was realised that the expense and delay occasioned by such a procedure could not normally be tolerated in design practice. More recently [6], a method of calculating the shear behaviour of complete roof panels has been advanced and satisfactory agreement has been obtained with experimental results [6, 7].

Before considering in detail the shear behaviour of roof decks and the analysis of an actual building allowing for the contribution of the decking, it is important to first consider whether it is safe or even desirable to take this stiffening effect into account in design.

Safety of Clad Buildings

The prime purpose of most shed-type buildings is to act as an umbrella to protect the contents from the weather, so that much of the load on the building frames is dependent on the sheeting (i. e. dead load, wind load and snow load). If the sheeting were totally removed much of the load would also be removed. Under such conditions, it is suggested that the effect of the sheeting should be taken into account in design. If the major part of the load on the building is derived from other sources, then the argument for allowing for the sheeting is not so strong.

If cladding is to be taken into account, it must be regarded as a structural element and proper care must be taken in specifying fixings, etc. For instance, hook bolts would not suffice, but self tapping screws or fired pins would be necessary. There is also the difficulty that sheeting has traditionally been regarded as an element which can be easily removed for maintenance purposes. Hence, it could be asked whether unauthorized removal of a number of sheets in an important part of the building would endanger the structure. It might also be asked if deterioration of the sheeting could weaken the whole building rather than cause a local weakness.

The foregoing questions are quite legitimate and contain elements of truth. Undoubtedly, if the membrane strength of cladding is to be used in design, then it will be necessary to train engineers to think in terms of the whole building, rather than in terms of a framework. Nevertheless, this has been done in the unitary construction of car bodies, in the stressed skin construction of airframes and in the design of ships' hulls, so there seems no basic reason why it should not be done in the structural industry, in spite of the special problems.

Because of the varied workmanship likely to be achieved in the site fixing of cladding, it is necessary to have proper safeguards. It is also necessary to ensure that the building is safe at all stages of construction, occupation and use. It is therefore suggested that the bare steel framework must be strong enough to withstand by itself all the design loads but that the maximum stress under this condition be allowed to approach the yield stress. After sheeting, the maximum calculated stress in the clad frame under the design loads should not exceed the present permissible working stress, and the calculated deflexions should be acceptable. By conforming to these conditions the safety of a building would be assured and economy in the design of the frame would result. It would also mean that the design reflected the true behaviour of the building rather than the hypothetical behaviour based on the bare frame.

Shear Tests on Steel Decks

In order that the method proposed for calculating the shear behaviour of panels of sheeting or decking may be assessed, the results of three sets of tests are compared with the calculated values. These tests are described more fully in references [6] and [7]. The method of calculation is given in reference [6] and illustrated by the example in the present paper.

Test 1. This test was carried out on a panel of steel sheeting 8 ft. wide × 10 ft. deep with 3 purlins (Fig. 1). The sheeting was 0.024 in. thick, the pitch of the corrugations was 4 in., the seam bolts were spaced at 12 in. centres and the sheet-purlin fasteners (self tapping screws) were spaced at various centres. Table 1 gives a comparison of the calculated and measured shear flexibilities and shear strengths.

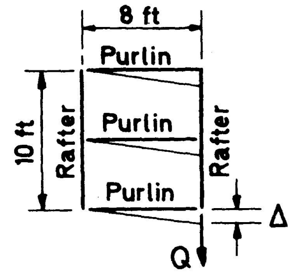


Fig. 1. Panel of sheeting in test 1.

Table 1

No. of fasteners per sheet	Shear flexibility Δ/Q in in./ton		Shear strength in tons	
	Calculated	Measured	Calculated	Measured
7	0.18	0.17	1.7	2.3—2.8
4	0.27	0.31	1.7	1.6—1.8
3	0.40	0.38	1.4	1.0—1.6

Test 2. The panel of steel sheeting was 12 ft. × 12 ft. with provision for either 3 purlins or 5 purlins (Fig. 2). The sheeting was 0.028 in. thick, the pitch of the corrugations was $6^3/_4$ in., the seam fasteners (pop rivets) were generally at 18 in. centres and the sheet-purlin fasteners (self tapping screws) were

fixed in every corrugation or alternate corrugations. Table 2 summarizes the calculated and observed behaviour.

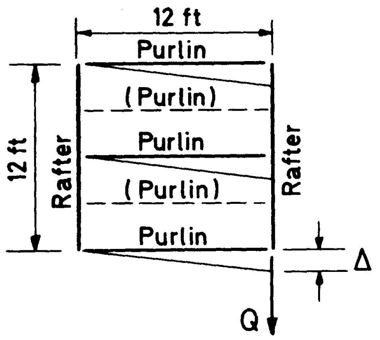


Fig. 2. Panel of sheeting in test 2.

Table 2

No. of purlins	Sheet-purlin fasteners	Shear flexibility Δ/Q in in./ton		Shear strength in tons	
		Calculated	Measured	Calculated	Measured
3	Every corrug.	0.060	0.056	3.5	3.4
	Altern. corrug.	0.359	0.328		
5	Every corrug.	0.045	0.050		
	Altern. corrug.	0.305	0.237		

Test 3. The panel was of steel sheeting 5 m wide × 3 m deep with 4 light gauge purlins (Fig. 3). The sheet thickness was 0.85 mm, the pitch of the corrugations was 125 mm, the seam fasteners (pop rivets) and the sheet-purlin fasteners (self tapping screws) were spaced at 125 mm or 250 mm centres. Table 3 summarizes the calculated and observed behaviour of the original and modified panels.

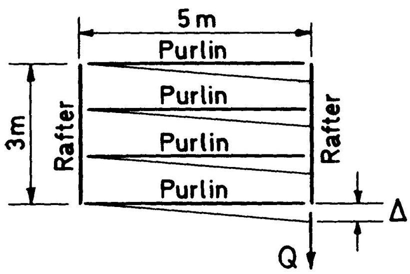


Fig. 3. Panel of sheeting in test 3.

Table 3

	Spacing of sheet-purlin fasteners	Spacing of seam fasteners	Shear flexibility Δ/Q in in./ton	
			Calculated	Measured
Original	250 mm	250 mm	0.97	1.30
Panel	125 mm	125 mm	0.91	1.36
Modified	125 mm	125 mm	0.10	0.14
Panel	250 mm	125 mm	0.14	0.15

Summary of Results. The tests show that the calculated values of the shear flexibilities of the panels tested are generally in satisfactory agreement with the measured values. The shear strengths in Tests 1 and 2, calculated on the assumption that failure occurs due to tearing at the sheet fasteners, are also in approximate agreement with the observed values. A comparison of shear strengths in Test 3 was not possible as failure did not occur in the above mode.

On the evidence of the above results, and others not quoted, it would appear that a reasonable estimate of the shear flexibility of panels of steel sheeting can be made provided that the details of construction do not differ too widely from the panels tested. Also, an estimate of the shear strength can be made assuming that failure occurs by tearing at the sheet fasteners (the usual mode).

Analysis of Clad Building

The steel framed building in question was 80 ft. wide, 309 ft. long and 50 ft. high. A diagrammatic representation of the main frames is shown in Fig. 4 and the plan of the building is given in Fig. 5. It is seen that the trans-

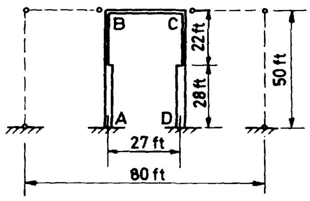


Fig. 4. Idealized steel frame.

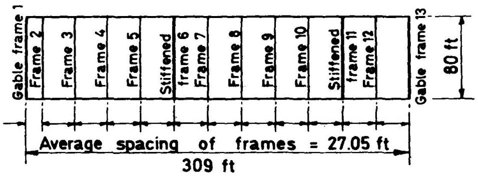


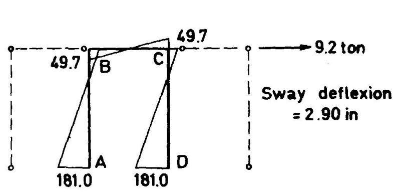
Fig. 5. Plan of roof deck.

verse strength of the building depends on the rigid portal frame ABCD in Fig. 4 and that the outer pinned frames, shown dotted, do not contribute to this strength. Referring to Fig. 5, there are rigid partition walls across the building at frames 6 and 11, so that the greatest length of building to be considered is the portion between these frames. In this portion, there are four intermediate frames and the average width of a panel of sheeting is 27.05 ft.

Bare Frame Analysis

In the bare frame analysis, the bending moments due to wind loads far exceeded those due to any other type of loading. In order to simplify the example, only wind bending moments will therefore be considered. Fig. 6 shows these calculated bending moments in the bare steel frame under working loads; the calculated sway deflexion is 2.90 in.

Fig. 6. Bare frame bending moments in ton-ft.



Shear Flexibility of Panel of Sheeting

The flexibility of a panel of sheeting will be due to:

1. 1.1. Bending of the corrugation profile.
   1.4. Shear strain in the panel.
   1.5. Axial strain in the purlins.
2. 2.1. Slip in the sheet/purlin fasteners.
   2.3. Slip in the seam fasteners.
3. 3.1. Twisting of the purlin/rafter connexions.

The above sub-heading numbers are those used in reference 6 where an expression for each effect was separately derived.

1.1. *Bending of the corrugation profile.* The sheet profile is shown in Fig. 7.

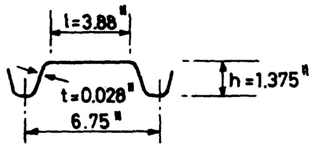


Fig. 7. Corrugation profile.

The shear flexibility due to distortion of the profile is given by

$$c_{1.1} = \frac{144\,K\,h^3\,l^2}{E\,t^3\,b^3}\,n_c\,f_1 \times 13.8,$$

where b = depth of panel = 80×12 in.
E = modulus of elasticity = 13,000 ton/in.^{2}
h = height of corrugation = 1.375 in.
l = width of flat top of corrugation = 3.88 in.
t = thickness of sheeting = 0.028 in.
K = constant of sheeting (ref. [7], table 4) = 0.145
n_c = number of corrugations per panel = 48
f_1 = reduction factor to allow for the effect of intermediate purlins (ref. [6], table 1) = 0.49
13.8 = multiplication factor to allow for fasteners in alternate corrugations (ref. [7], table 4)

hence $c_{1,1} = 1.0 \times 10^{-3}$ in./ton (1)

1.4. *Shear strain in panel.* The shear flexibility due to distortion of the panel from a rectangle to a parallelogram is given by

$$c_{1.4} = \frac{2\,a\,(1+\nu)}{b\,t\,E}\,\frac{\text{developed length of profile}}{\text{pitch of corrugations}}\,f_2,$$

where a = average width of panel = 27.05×12 in.
ν = Poisson's ratio = 0.25
developed length/pitch = 1.26 (ref. [7], table 4)
f_2 = reduction factor to allow for the effect of intermediate purlins (ref. [6], table 1) = 0.29

hence $c_{1.4} = 0.8 \times 10^{-3}$ in./ton (2)

1.5. *Axial strain in purlins.* The shear flexibility due to the tendency of the purlins to lengthen or shorten under axial stress is given by

$$c_{1.5} = \frac{2\,a^3}{3\,b^2 A\,E} f_3,$$

where A = cross sectional area of purlins = 2.68 in.^{2}
f_3 = reduction factor to allow for the effect of intermediate purlins (ref. [6], table 1) = 0.39

hence $c_{1.5} = 0.3 \times 10^{-3}$ in./ton (3)

2.1. *Slip in sheet/purlin fasteners.* The shear flexibility due to this cause is given by

$$c_{2.1} = \frac{2\,s\,p}{a}\left[\frac{6}{n_p} + \frac{a^2 f_3}{b^2}\right],$$

where p = pitch of fasteners = 13.5 in.
s = slip of fastener per unit load (ref. [6]) = 0.10 in./ton
n_p = number of purlins = 13

hence $c_{2.1} = 4.2 \times 10^{-3}$ in./ton (4)

2.3. *Slip in the seam fasteners.* The shear flexibility due to seam slip is given by

$$c_{2.3} = \frac{n_{sh}\,s_s}{n_s},$$

where n_{sh} = number of sheet widths per panel = 14
n_s = number of seam fasteners per seam = 54
s_s = slip of seam fastener per unit load (assumed to have the same value as s) = 0.10 in./ton

hence $c_{2.3} = 25.9 \times 10^{-3}$ in./ton (5)

3.1. *Twisting of the purlin/rafter connexions.* Fig. 8 shows a typical purlin/rafter connexion at eight of the purlins; the remaining connexions were more

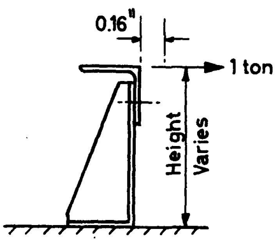


Fig. 8. Detail of purlin/rafter connexion.

flexible. From tests on other types of connexion (ref. [6]), it can be estimated that the flexibility per connexion is 0.160 in./ton.

hence
$$c_{3.1} = \frac{0.160}{8} = 20.0 \times 10^{-3}\,\mathrm{in/ton}. \tag{6}$$

Total Shear Flexibility

From Eqs. (1) to (6), the total shear flexibility is given by

$$c = (1.0 + 0.8 + 0.3 + 4.2 + 25.9 + 20.0) \times 10^{-3} = 0.052\ \mathrm{in./ton}.$$

Modified Bending Moments and Deflexions in Clad Frames

From Fig. 6, the flexibility of a bare frame is

$$k = \frac{\text{sway deflexion}}{\text{sway force}} = \frac{2.90}{9.2} = 0.315\,\mathrm{in/ton},$$

Hence, the relative stiffness factor r is

$$r = \frac{c}{k} = \frac{0.052}{0.315} = 0.168.$$

Instead of using the type of design chart derived in reference [5], the information is tabulated. It is seen from Table 4 that for a building with 4 intermediate frames and for $r = 0.168$, the maximum value of m is 0.34. This is used in the expression

Final moment in clad frame =
Non sway moment + m × Sway moment of bare frame.

Since only wind bending moments are being considered, the non sway moment is zero and

Final moment in clad frame = 0.34 × Sway moment in bare frame.

This expression applies to the two central frames of the portion considered, i. e. frames 8 and 9. The relevant factor for frames 7 and 10 is 0.24. For frames 8 and 9 the modified bending moment diagram is given in Fig. 9; the sway deflexion is 0.34 of the bare frame value.

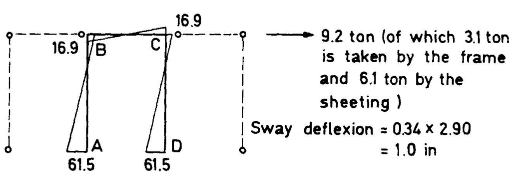


Fig. 9. Clad frame bending moments in ton-ft.

Table 4. Reduction factors to be applied to sway moments in clad buildings with 4 intermediate frames

Relative stiffness factor r	Reduction factor m to be applied	
	Intermediate frames 1 and 4	Intermediate frames 2 and 3
0.02	0.04	0.06
0.04	0.07	0.11
0.06	0.10	0.16
0.08	0.13	0.20
0.10	0.16	0.24
0.12	0.19	0.27
0.15	0.22	0.32
0.20	0.27	0.39
0.25	0.31	0.45
0.30	0.35	0.50
0.40	0.41	0.58
0.50	0.46	0.64
0.60	0.49	0.68
0.80	0.56	0.75
1.00	0.60	0.80

Forces on Roof Sheeting

Referring to Figs. 5 and 9, the force on the sheeting at frames 8 and 9 is $(1-0.34) \times 9.2 = 6.1$ tons and the force on the sheeting at frames 7 and 10 is $(1-0.24) \times 9.2 = 7.0$ tons. Hence the total shear force on the sheeting between frames 6 and 7 and between frames 10 and 11 is 13.1 tons. This represents an average shear stress in the sheeting of $13.1/80 \times 12 \times 0.028 = 0.5$ ton/in.^{2}, which is very small compared with the likely stress due to ordinary flexure of the sheet.

Ultimate Shear Strength of Panel

The strength of the panel will be calculated on the assumption that failure occurs due to

1. Tearing at the sheet/purlin fasteners.
2. Failure of the seam fasteners.

1. *Tearing at the sheet/purlin fasteners.* The normal forces on the fasteners are assumed to vary linearly along the length of a purlin as shown in Fig. 10.

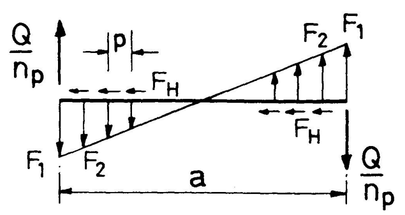


Fig. 10. Forces acting on a purlin.

The maximum normal force on a fastener, F_1, can be shown to be given by

$$F_1 = \frac{Q}{n_p\left[1+\left(\frac{a-2p}{a}\right)^2+\left(\frac{a-4p}{a}\right)^2+\cdots\right]} = \frac{Q}{13\times 4.46} = 0.017\,Q\,.$$

The force per fastener along the purlin, F_H, is given by

$$F_H = \frac{Q}{b}\,p\,f_3 = 0.005\,Q\,.$$

Hence, the maximum resultant load per fastener is

$$F_r = \sqrt{F_1^2+F_H^2} = 0.018\,Q\,.$$

From tests, the ultimate tearing load per fastener $= 0.42$ tons, so $Q = 0.42/0.018 =$ 23.4 tons which is the maximum shear strength of a panel according to this criterion.

2. *Failure of the seam fasteners.* The total shear forces on the purlins give rise to a moment $Q a$ (Fig. 11) which is resisted by the forces on the sheeting. Hence the total shear force distribution across the sheeting is as shown in Fig. 12, the maximum value being $\frac{3}{2}\,Q$.

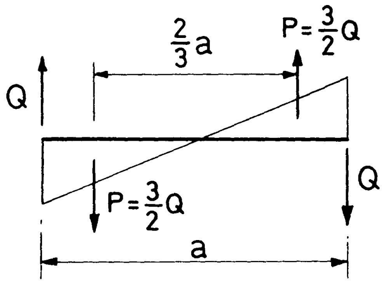


Fig. 11. Total forces acting on purlins.

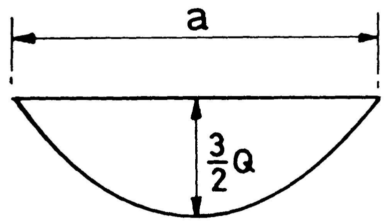


Fig. 12. Shear force distribution across sheeting.

Along the central seam there are 40 screws (ultimate tearing load 0.34 ton each) and 14 self tapping screws into the purlins (0.42 ton each) so that the maximum allowable shear force is $40\times 0.34+14\times 0.42 = 19.5$ ton. Equating this to $\frac{3}{2}\,Q$ gives the maximum permissible shear strength of a panel as $Q = 13.0$ ton.

From criteria (1) and (2), it is apparent that case (2) dominates. Hence the maximum shear force, under working loads, in a panel of sheeting, is slightly greater than that to cause tearing at the seam fasteners (13.1 ton cf. 13.0 ton).

Conclusions

From the calculations for a particular clad building, it is shown that the sway bending moments and deflexion may be drastically reduced by allowing for the effect of sheeting in design. Conversely, it can be said that in analysing the given building, unless the cladding is taken into account, the calculated stresses are fictitious.

In the building considered, the seam fasteners in certain panels of roof decking are on the point of tearing the sheeting, even under working loads. Obviously this does not endanger the structure, since the steelwork has been designed on the basis of bare frames, but it could cause trouble in keeping the cladding watertight. The level of shear stress in the sheeting is so low that design of the sheeting would be determined by the ordinary flexural requirements.

By using the effect of sheeting in design it should be possible to achieve greater economy than at present without loss of safety, for the design would be based on the actual behaviour of the building rather than on the hypothetical behaviour of the bare frame.

Acknowledgements

Thanks are due to the Science Research Council for supporting the work being carried out at the University of Manchester into the stiffening effect of sheeting in buildings.

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Summary

A review is given of recent work on the diaphragm behaviour of light steel sheeting, with particular reference to shed-type buildings. The safety and desirability of taking account of the stiffening effect of sheeting in structural design is then considered.

A resume is given of three sets of shear tests on steel sheeting and decking and the results are shown to be in satisfactory agreement with the calculated values. On the basis of this agreement, a specimen calculation is made of the stiffening effect of sheeting on an actual steel framed building. The calculations show that the actual wind bending moments and transverse deflexions will be only one third of the bare frame values, and that the seam fasteners in the end panels of roof decking are stressed to the limit, even though the building was not designed to take account of the cladding.

Résumé

L'auteur commence par un bref aperçu sur les travaux récents concernant le comportement de diaphragmes en panneaux d'acier minces, où l'accent est mis sur les constructions à sheds. Puis il étudie la sécurité et l'utilité de la prise en considération de la rigidité des panneaux dans le dimensionnement d'une construction.

Il résume trois séries de tests de cisaillement faits sur des panneaux de façades et de toitures en acier. Leurs résultats se rapprochent de façon satisfaisante des valeurs calculées. Cette concordance permet de développer un calcul type pour évaluer l'effet de renforcement dû aux panneaux sur une construction d'acier à portiques. Les calculs montrent que les valeurs réelles des moments de flexion et des déplacements transversaux dus au vent se réduisent à un tiers des valeurs calculées sur le squelette nu, et que les fixations des panneaux extérieurs du toit sont sollicités à la limite, bien que le bâtiment était projété en négligeant les revêtements.

Zusammenfassung

Die Verfasser geben einen Überblick über die Scheibenwirkung von Leichtstahlplatten mit besonderer Berücksichtigung von Gebäuden mit Schirmdach (Shed-Dach).

Sicherheit und Wunsch, dem Steifigkeitseinfluß der Platten Rechnung zu tragen, werden sodann untersucht. Für drei Schubversuchssätze von Stahlplatten und -decken wird eine Zusammenfassung angegeben, und es zeigt sich, daß die Ergebnisse mit den errechneten Werten gut übereinstimmen.

Auf Grund dieser Übereinstimmung wurde eine spezielle Berechnung über den Steifigkeitseinfluß der Platte für einen gängigen Stahl-Stockwerkrahmen angestellt. Die Rechnung zeigt uns, daß die wirklichen Windbiegemomente und Querverschiebungen nur einen Drittel der Werte des baren Rahmens ausmachen und daß die Saumverbindungen in den Endfeldern der Dachplatten bis zur Grenze beansprucht sind, als ob das Gebäude ohne Verkleidung entworfen worden wäre.