Zeitschrift: IABSE publications = Mémoires AIPC = IVBH Abhandlungen

Band: 36 (1976)

Artikel: Light gauge steel folded plate construction

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DOI: https://doi.org/10.5169/seals-923

Light Gauge Steel Folded Plate Construction

Toits plissés formés d'éléments minces en acier

Faltwerke aus Stahlleichtprofilen

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Abstract

The alternative types of light gauge steel folded plate structures are briefly described and the current state of the art reviewed. Previous work in this subject, including a considerable number of actual structures, has been almost exclusively in the USA and has utilised welded construction. The fabrication of such structures using a single skin of corrugated sheeting and conventional sheeting fasteners such as self-tapping screws clearly presents an attractive possibility and the investigation described in this paper is directed towards demonstrating the viability of this approach.

The analysis and design of light gauge steel folded plates is dependent on the readily justifiable assumption that plate and slab action may be separated and revolves critically around the prediction of the behaviour of individual plate elements. An extensive test series, based on the full scale testing of a range of plate elements of 17.6 m span and 2.54 m depth is described and results for both strength and stiffness are compared with available theories. It is concluded that this form of design is readily and economically possible for spans up to and possibly exceeding 20-25 m. The limiting factors are shear buckling of the sheeting which is largely independent of the mode of fastening and deflection which is greatly influenced by the distribution of fasteners.

Various practical considerations are discussed including erection of folded plate roofs and the possibility of particularly favourable sheeting arrangements purpose-made for folded plate construction.

Introduction

A typical light gauge steel folded plate roof is shown conceptually in Figure 1. Apart from gable framing the roof structure has only two primary elements, namely longitudinal fold line members and sheeting spanning between them. NILSON [1, 2] showed some years ago that the design of such a structure could be attractively simple and as a result of his work a variety of folded plate structures have been built in the USA [3, 4, 5, 6].

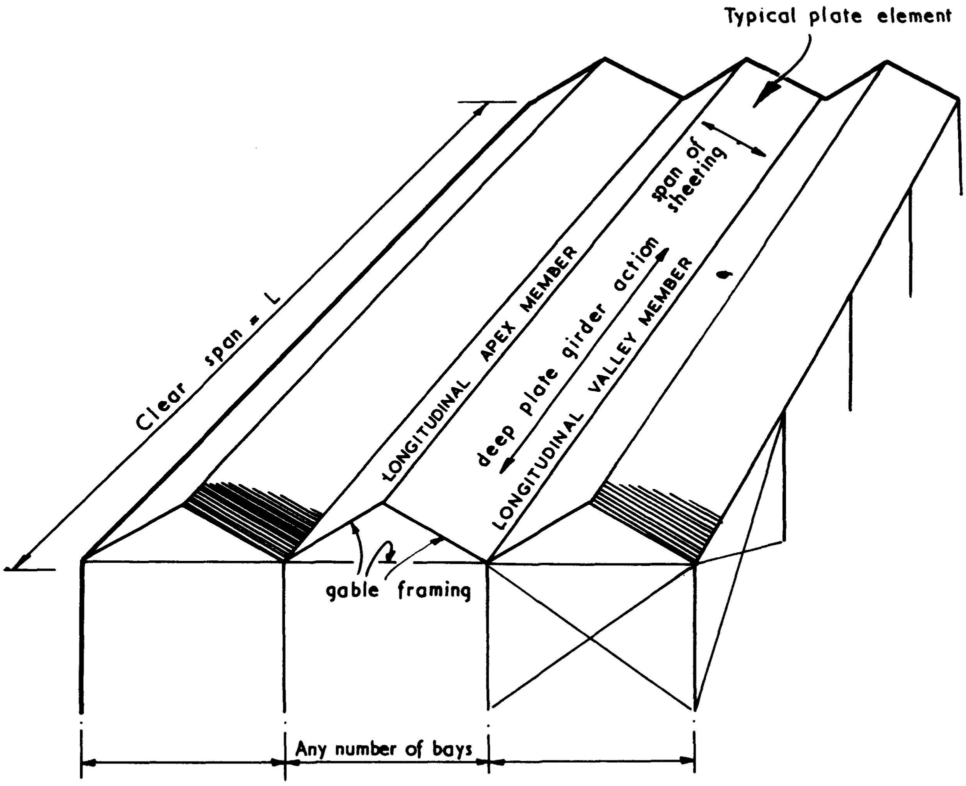

Fig. 1. Concept of Light Gauge Folded Plate Roof.

Little of fundamental importance has been added since Nilson's early investigations although FALKENBURG [7] has proposed some interesting alternative shapes and given detailed consideration to the connection between corrugated steel sheeting and the fold line members and SCHOELLER et al. [8] have utilised finite elements for light gauge steel folded plate analysis.

Nilson demonstrated a number of basic principles applicable to the design of light gauge steel folded plates, namely:

(1) Panels of corrugated steel sheeting span between fold line members so that uniformly distributed loads on the roof appear as line loads at the fold lines as far as overall behaviour is concerned. The almost complete lack of bending rigidity transverse to the corrugations means that the sheets span one way only; there is no redistribution of load along the panel.

(2) Line loads on the fold lines resolve themselves into in-plane loads on the two plate elements which meet at a given fold line. Out-of-plane bending stiffness of complete plate elements can be neglected when compared with the considerable in-plane stiffness so that it is merely necessary to resolve the line load into two components in the planes of the plates. A necessary consequence of the lack of out of plane stiffness is that both edges of each individual plate element

must be stiffened either by another plate element meeting at an appreciable angle or by some other means such as an edge beam or vertical cladding.

(3) It follows that the force system acting on an individual plate element is statically determinate and consists solely of an in-plane distributed load. Thus the basic design problem is as shown in Figure 2 and it is necessary to predict the strength and stiffness of such typical plate elements.

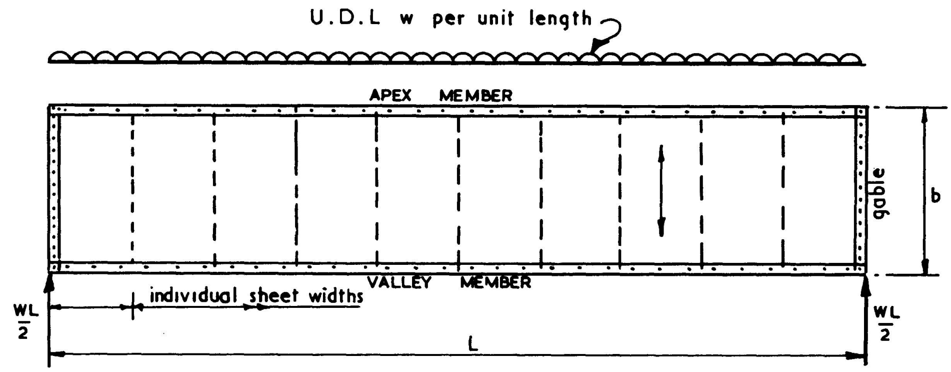

Fig. 2. Typical Plate Element.

(4) These in-plane forces give rise to bending moments and shear forces in the individual plate elements. The bending moments give rise to axial forces in the fold line members, the shear forces are carried entirely in the sheeting and the two effects may be considered independently.

Thus the complete roof acts as a series of interconnected deep plate girders spanning between gable framing which must be capable of carrying the roof loads to the foundations.

(5) Provided that the deflection of the individual plate elements can be estimated the deflection of the complete roof can be derived by considering simple displacement diagrams at the fold lines.

It may also be mentioned that a wide range of alternative structural arrangements are possible. For instance, although the simple saw-tooth arrangement is the most efficient other cross-sections are feasible such as those shown in Figure 3 and plate elements may be tapered giving rise to more interesting shapes such as the pleated dome shown in Figure 4.

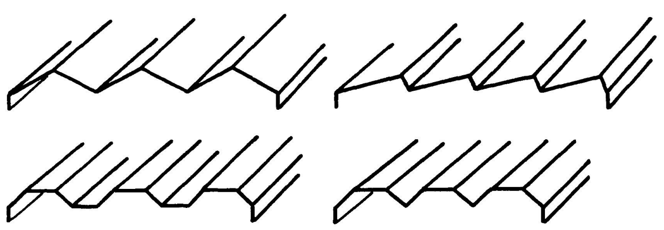

Fig. 3. Some Alternative Cross-sections for Folded Plate Structures.

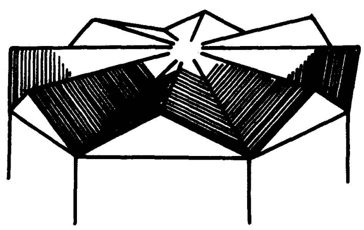

Fig. 4. Pleated Dome Structure.

Because many such structures exist, the principles underlying the use of light gauge steel sheeting as the shear-carrying component of a folded plate roof must be considered to be sound. However, previous practice has typically utilised welded construction with a double skin of sheeting resulting in an extremely stiff and strong structure in which neither distortions or deflections are usually significant.

No attempt has previously been made to form the plate elements more economically by applying the constructional practice which is more conventional in Britain, namely using a single skin of corrugated steel sheeting with discrete fasteners such as fired pins or self-tapping screws. The investigation reported in this paper is part of a program of research which is being carried out in order to establish the feasibility of this latter form of construction and to demonstrate that both the strength and the stiffness of such folded plate roofs may be predicted from simple expressions. As the crucial factor is the in-plane behaviour of individual plate elements, the investigation has concentrated to date on this aspect which is more readily amenable to full-scale laboratory investigation.

Expressions for the flexibility and strength of individual plate elements subjected to uniformly distributed load have been derived and justified by finite element analysis [9]. They have also been compared [9] with the results of tests on plate elements of very thin sheeting (0.46 mm thickness and 19 mm depth). Very good correlation of experimental and theoretical deflections was obtained but it was only possible to obtain very limited information regarding failure loads as a result of the very low buckling capacity of the sheeting used.

This paper describes how the test series was continued using a stronger sheeting which is typical of the lightest sheeting likely to be used in practice. The test arrangement, which simulated the uniformly distributed load by a series of point loads, is first described. The appropriate forms of the design expressions for this form of loading are then given, thus enabling a comparison of experimental and theoretical results to be made.

Finally some of the practical issues involved in the design and construction of complete folded plate structures are considered. It is concluded that such structures are a most attractive possibility both aesthetically and economically and that spans in excess of 20 metres are readily possible using conventional sheeting and fasteners.

Tests on Full Scale Panels

In order to initiate this test series, a specific situation was considered and a plate element chosen as a typical element of a complete folded plate roof as shown in Figure 1. In order to suit the fixing points in the laboratory floor, the dimensions shown in Figure 5 were adopted. It was assumed that the roof slope was 36.9° (3-4-5 triangle) and that the total vertical loading on the roof was 0.7 kN/m^2. For this loading, British Steel Corporation "Longrib 700" sheeting was adequate to span simply supported between the fold line members and was adopted for the tests. The profile of this sheeting was as shown in Figure 6. The sheeting was galvanised and had a total thickness of 0.7 mm. The net thickness was taken to be 0.66 mm for the purpose of theoretical comparison. The sheeting was fixed with seams in the crests of the corrugations.

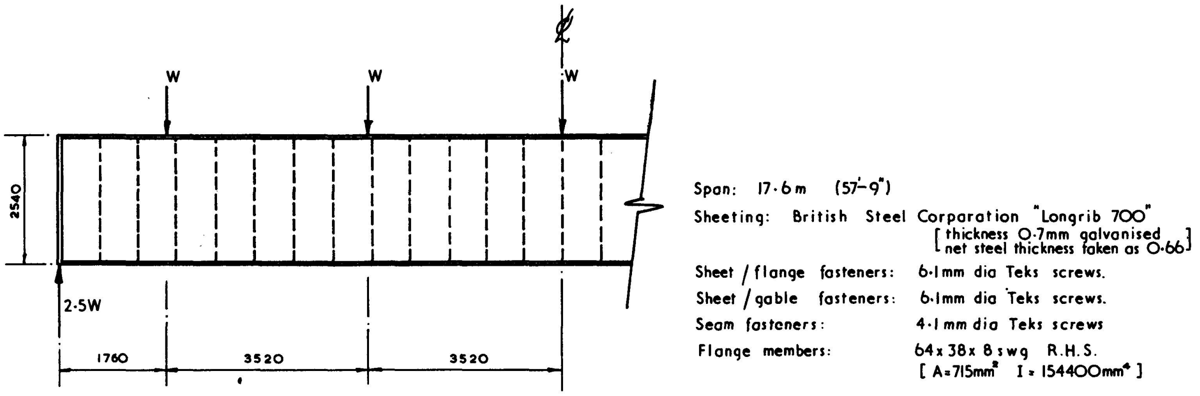

Fig. 5. General Arrangement of Test Panels.

In order to facilitate the loading of the test panel by five hydraulic jacks simulating the uniformly distributed load, the flange members were chosen from the available range of rectangular hollow sections. 64 mm × 38 mm × 8 SWG (2½" × 1½") RHS were found to be adequate to carry the flange forces and were adopted. With the benefit of hindsight it is evident that more useful information regarding failure loads would have been obtained if a section with greater resistance to local bending from the jack forces had been chosen. Even with spreader beams and flexible packing there was a tendency for extensive local distortion in the vicinity of the jacks to occur before the failure load of the panel could be attained. Thus although a total of nine different tests were carried out only two of these were carried through to failure the remainder being either confined to the working load range or terminated before failure due to excessive local distortion at the loading points.

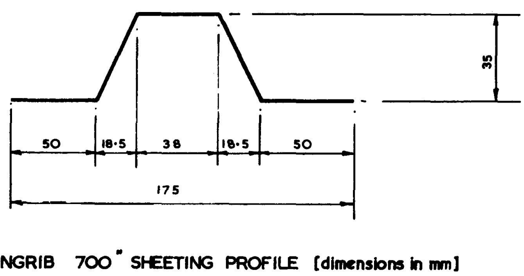

Fig. 6. "Longrib 700" Profile.

With the dimensions and details described above, the working load was 8.4 kN per jack. Tests were carried out using a variety of arrangements of fasteners giving (with the exception of test No. 8) theoretical factors of safety against fastener failure of the order of 2.0 and giving a reasonable range of panel flexibilities. The entire test series is described in table 1, the meaning of the symbols defining the fastener arrangements being shown in Figure 7 and Appendix A.

Figure 8 shows a typical test in progress. It may be observed that the compression flange was prevented from buckling laterally by suitable restraints incorporating needle bearings to eliminate friction. The only measurements taken during loading were deflection readings opposite the loading points.

Table 1. Summary of fastener arrangements and experimental results

Test Number	Pitch of flange fasteners (mm) p_1	p_2	p_3	Number of gable fasteners n_{sc}	Number of seam fasteners $n_1=2$ n_{s1}	$n_2=5$ n_{s2}	$n_3=5$ n_{s3}	Test Results: Central deflection (mm)	Test Results: Ultimate Load (kN)	Theoretical Predictions: Initial Deflection (mm)	Reloading Deflection (mm)	Ultimate load (kN) and mode of failure
1	175			18	24		12	2.13 / 1.63		1.94	1.76	
2	175			18	24		6	1.64		1.97	1.77	
3	175			18	24	16	6	1.73 / 1.84	16.1	2.01	1.78	18.1 (seam a-b)
4	175			10	24	16	6	1.71		2.02	1.78	
5	175		350	10	24	16	6	2.31		2.81	2.57'	
6	175	350		10	24	16	6	4.54		5.22	4.94	
7	175	350	525	10	24	16	6	5.18		6.27	5.97	
8	350			10	24	16	6	6.30 / 5.07		7.22	6.91	
9	175			10	49	26	12	1.64	16.7	1.92	1.76	17.8 / 14.2 (tension flange fasteners)

(Note: central deflections are per unit jack load, ultimate loads are load per jack)

In each test, load was applied in increments of 1 kN at each jack and deflection readings taken. The load deflection curves were linear up to the working load and from the slope of these curves the test results for central deflection given in table 1 were obtained.

Two tests were continued to failure. In test 3 a seam failure was obtained adjacent to the left hand support as shown in Figure 9. In test 9, the tension flange fasteners tore first followed immediately by buckling of the sheeting as shown in Figures 10 and 11. The excessive number of seam fasteners shown in Figure 11 served solely to ensure that a different mode of failure to that of test 3 was obtained.

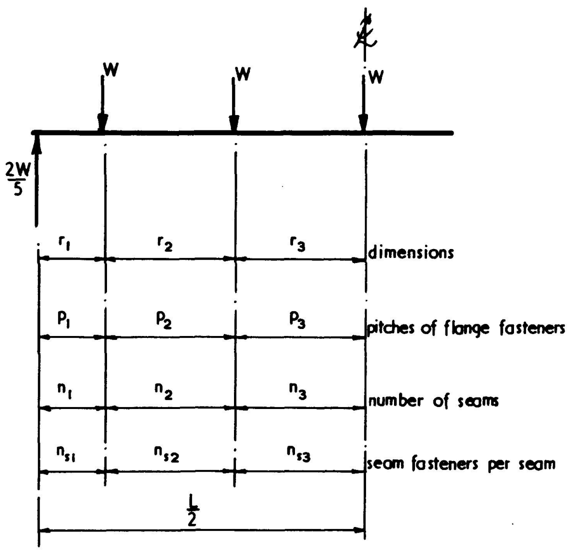

Fig. 7. General Arrangement of Fasteners for Theory.

Table 2. Factors for distribution of flange fastener forces

n_f	g_1 seam fasteners in crests	g_1 gable fasteners in troughs
2	0.13	1.00
3	0.30	1.00
4	0.44	1.04
5	0.58	1.13
6	0.71	1.22

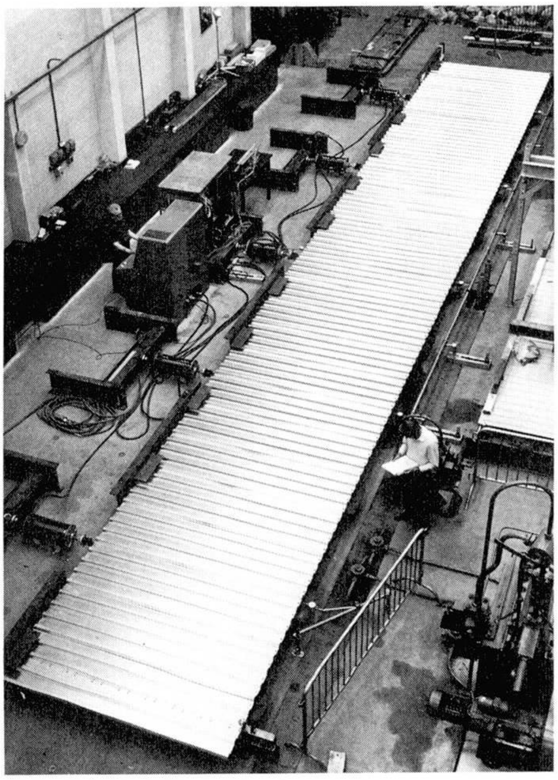

Fig. 8. Typical Test in Progress.

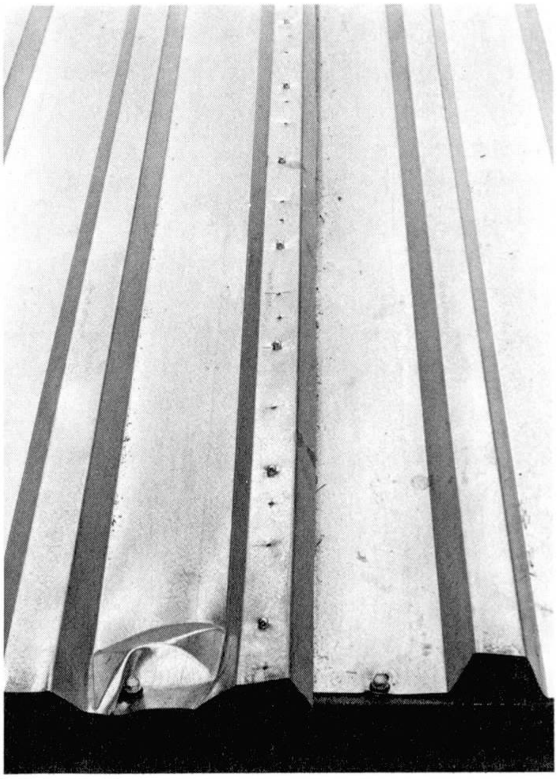

Fig. 9. Seam Failure – Test 3.

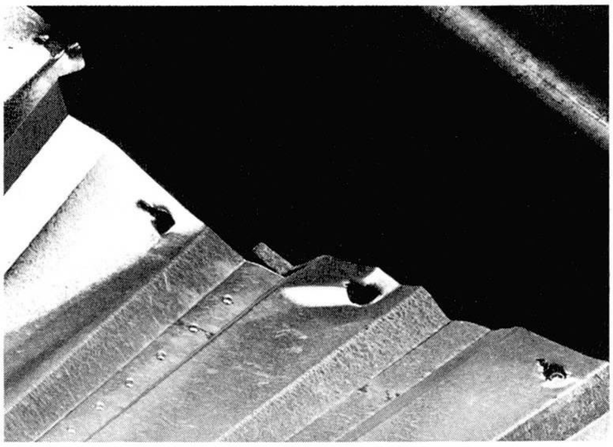

Fig. 10. Tearing of Flange Fasteners – Test 9.

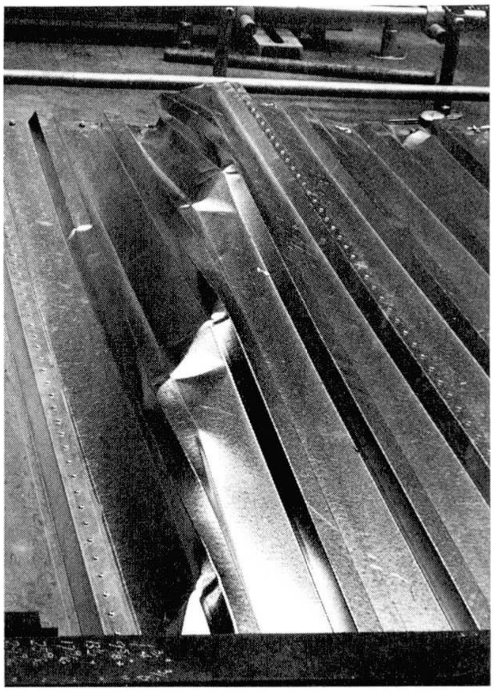

Fig. 11. Illustrating Overall Buckling Following Fastener Failure – Test 9.

It may be noted that in table 1, two values of deflection are given for tests 1, 3 and 8 each of which was repeated. It has been found that during the testing of both shear diaphragms and folded plate elements the initial loading is always more flexible than subsequent loadings as a consequence of fasteners "bedding in". This effect is very noticeable in reloading tests on individual fasteners. The higher of the two central deflection results shown for tests 2 and 8 were both obtained for panels that had been completely refastened before the test and therefore represent the only genuine results for initially unloaded panels. The remaining results all reflect some degree of prior loading. In order to accommodate this state of affairs, theoretical predictions based on both the initial and reloading flexibility of the fasteners are included in table 1. It can be seen that the difference is not marked as much of the theoretical flexibility is in the distortion of the corrugation profile and in axial strain in the flange members, fastener flexibility playing a comparatively minor role.

Theoretical Expressions for Strength and Deflection

As the light gauge folded plate structure offers an efficient means of spanning considerable distances, one of the prime objectives of the design process must be the accurate prediction of deflections which often tend to be large when light loadings are carried over long spans. The expressions that follow are more readily derived from the work of BRYAN [10] on shear diaphragms as subsequently modified by the author [11] than from the expressions for plates subject to uniformly distributed loading [9]. They are derived by considering assumed internal force distributions which have been found to be very close to those found by comprehensive finite element analysis and have given very good results for shear diaphragms.

Expressions for failure load arise directly out of the assumed internal forces. Expressions for deflection arise by assuming that the total deflection to be the sum of the following components which may be considered separately:

$d_{1.1}$ distortion of the corrugation profile of the sheeting.
$d_{1.2}$ shear strain in the sheeting.
$d_{1.3}$ bending action causing axial strains in the flanges.
$d_{2.1}$ slip in the sheeting/flange fasteners.
$d_{2.2}$ slip in the seam fasteners.
$d_{2.3}$ slip in the sheeting/gable member fasteners.

The detailed derivations will not be given as in most cases the expressions can be written down directly, the only exception being $d_{1.3}$ which requires some elementary structural mechanics. The notation used is defined in Appendix A and appropriate numerical values for the test series are given there.

Expressions for Strength

The expressions that follow arise from the consideration of various possible failure modes. The ultimate load of the structure is the lowest ultimate load obtained when all possible modes of failure are considered.

Sheet/flange Fastener Failure

$$W_{ult} = \frac{0.32\ bF_p}{p}$$

This expression incorporates an empirical reduction factor of 0.8 to allow for the influence of prying action at the fasteners as described in detail by FALKENBURG [7]. In the evaluation of the failure load for test 9, as shown in table 1, values with and without this factor are given for comparison purposes.

Seam Failure

$$W_{ult} = \frac{(n_s s + g_1 s_s) F_s}{2.5\ s}$$

In this expression, g_1 is a tabulated factor for the distribution of sheet/flange fastener forces and is given for the case of "sheeting" (seam fasteners at the crest of the corrugations) in Table 2. The full significance of this factor is described in reference 9.

Gable Fastener Failure

$$W_{ult} = \frac{(n_{sc} s + 2 g_1 s_{sc})}{(n_{sc} s + 2 s_{sc})} \frac{(n_{sc} F_{sc} + 2F_p)}{2.5}$$

In this expression, the appropriate value of g_1 is that for "decking" (fasteners in the troughs of the corrugations) and is again given in Table 2.

Buckling Failure of Sheeting

Hlavecek's theory [12] has been found easiest to use but has generally appeared to require an arbitrary reduction factor (≥ 0.65) when applied to discretely fastened folded plate elements. As it is not directly relevant to the interpretation of the test results the appropriate expressions will not be reproduced herein.

Expressions for Deflection

The central deflection is found by summing the components $d_{1.1}$ to $d_{2.3}$.

Deflection due to Distortion of the Corrugation Profile of the Sheeting

$$d_{1.1} = \frac{0.144\ d^4}{Et^3 b^3} (2.5\ K_1 r_1 + 1.5\ K_2 r_2 + 0.5\ K_3 r_3)\ W$$

In this expression, r_1, r_2, r_3 are the dimensions of the regions of constant shear between the loads as shown in Figure 7 and K_1, K_2, K_3 are the corresponding values of the sheeting constant K which depends critically on the pitch of the sheet/flange fasteners. The appropriate values of K were obtained by finite element analysis over the correct sheet length and incorporating the influence of the supporting members [13]. The values used in the theoretical comparisons in table 1 were:

every corrugation fastened ($p = 175$) $K = 0.54$;
alternate corrugations fastened ($p = 350$) $K = 3.77$;
every third corrugation fastened ($p = 525$) $K = 8.01$.

Deflection due to Shear Strain in the Sheeting

$$d_{1.2} = \frac{2(1+\nu)(1+2h/d)}{Etb}(2.5\, r_1 + 1.5\, r_2 + 0.5\, r_3)\, W$$

Deflection due to Axial Strain in the Flanges

$$d_{1.3} = \frac{0.132\, L^3 W}{E\, A\, b^2}$$

Deflection due to Slip in the Sheeting/flange Fasteners

$$d_{2.1} = \frac{2s}{b^2}(2.5\, p_1 r_1 + 1.5\, p_2 r_2 + 0.5\, p_3 r_3)$$

Note. s represents the slip per unit load of the sheet/flange fasteners. Typical fastener slip characteristics for a series of simple lap joints in a similar thickness non-galvanised m.s. sheet are shown in Figure 12. The method of extracting s from these curves is [9] to take the average slip at half the average ultimate load leading to the nominal average slip of 0.076 mm/kN shown on the figure. Similar results for other thicknesses of sheet suggest that this may have been an unusually stiff value and a value of 0.15 mm/kN was used in the theoretical comparisons representing a reasonable upper limit of fastener flexibility. Based on a number of fastener reloading tests a lower limit of 0.01 mm/kN was also included in the theoretical comparisons.

Deflection due to Slip in the Seam Fasteners

$$d_{2.2} = s_s s \left[\frac{2.5\, n_1}{n_{s1} s + g_1 s_s} + \frac{1.5\, n_2}{n_{s2} s + g_1 s_s} + \frac{0.5\, n_3}{n_{s3} s + g_1 s_s}\right]$$

Note. Similar considerations apply to s_s, the slip per unit load at the seam fasteners, as were discussed for s above. Figure 13 shows typical characteristics for a similar non-galvanised sheet thickness. The values adopted for theoretical comparison were $s_s = 0.15$ mm/kN (initial loading) or 0.05 mm/kN (reloading).

Deflection due to Slip in the Gable Fasteners

$$d_{2.3} = \frac{2.5\, s_{sc} s}{n_{sc} s + 2 g_1 s_{sc}}$$

Note. Gable fasteners were identical to sheet/purlin fasteners so that: $s_{sc} = 0.15$ mm/kN (initial loading) or 0.01 mm/kN (reloading).

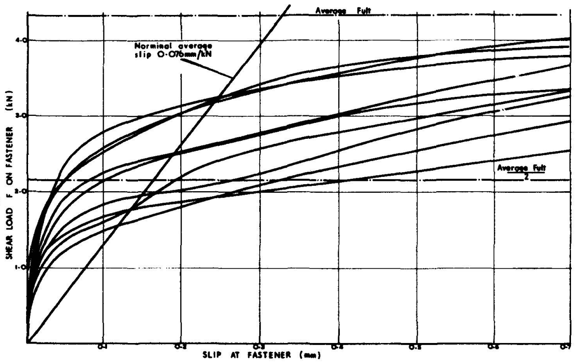

Fig. 12. Fastener Slip Characteristics for Barber Colman ¼-14 Tek Fasteners in 0.76 mm Thick Non-galvanised M.S. Sheet.

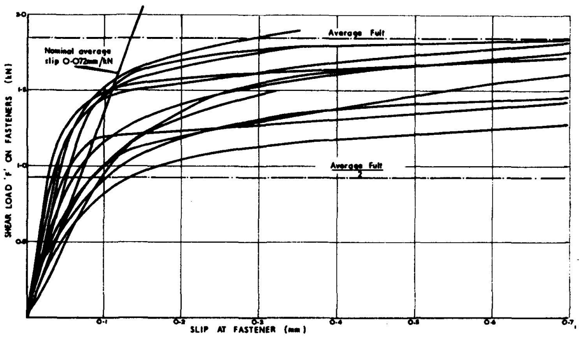

Fig. 13. Fastener Slip Characteristics for Barber Colman No. 8 Tek Fasteners in 0.76 mm Thick Non-galvanised M.S. Sheet.

Fastener Ultimate Loads

From a series of tests similar to those giving rise to Figures 12 and 13 the following values were adopted in calculating theoretical ultimate loads:

for sheet/flange fasteners	$F_p = 3.83$ kN
for seam fasteners	$F_s = 1.85$ kN
for sheet/gable fasteners	$F_{sc} = 3.83$ kN

Comparison of Experimental and Theoretical Results

The complete comparison summarising both deflection and strength is shown in table 1.

In almost every test, the experimentally obtained flexibilities fall slightly below the theoretical values, even when the reloading value of fastener flexibility is used. The differences become proportionally greater as the panels become more flexible. As this additional flexibility is due almost entirely to the increase in sheet distortion when the ends of the sheeting are fastened in alternate or every third corrugation, it may be concluded that the theoretical flexibilities used for these cases are on the high side. The reason for this is not completely clear and is under investigation. As any discrepancy is on the safe side it may be concluded that it is readily possible to predict adequately the deflections of discretely fastened folded plate roofs.

Relatively limited information is available regarding failure loads. The seam failure (test 3) was about 11% low which may well represent a slight deficiency in self-drilling self-tapping screws, when used in thin-to-thin connections at the thinner end of the range of sheeting thicknesses [9]. The sheet/flange fastener failure (test 9) lay between the limits of no reduction in strength due to prying action and the 20% reduction advocated for normal sheeting profiles [9] and this must be regarded as a satisfactory test result. The predicted buckling load was a value greater than 18.1 kN per jack and the mode of failure for test 9 may suggest that buckling was about to take place at a load not greatly in excess of the maximum achieved of 16.7 kN.

There is room for more test results for failure loads but it is already clear that all failure modes can be predicted with reasonable accuracy for failure at the gable fasteners cannot have any independent significance.

Erection of folded plate roofs

Up to date, the conventional method of erection has involved piecemeal erection on light internal scaffolding [4]. This is necessarily an expensive method and it may well be cheaper to build the roof either in its entirety or in sections on the ground and then to jack it into position using climbing jacks on the gable columns. A similar method was adopted for the hypars of the well known jumbo jet hangars [14] with great success. The lightness of construction would suggest that such an approach should prove ideal for more conventional folded plate structures and is to be adopted for prototype tests on a 21.6 m span folded plate roof at the University of Salford during 1976.

Some Practical Considerations in the Design of Folded Plate Roofs

Alternative Sheeting Arrangements

It must be recognised that conventional roof sheeting has a significant drawback when used in the context of light gauge steel folded plate construction. The cover width of commercially available sheets is generally less than one metre and as a result of considerable number of heavily loaded seams have to be fastened. This state of affairs suggests that there may be more satisfactory arrangements available, possibly with purpose made components. A number of alternative arrangements are recognised at this stage:

(a) *Wider sheets*

If the sheeting is formed in a press brake with the corrugations running across the width of the coil unlimited lengths of sheeting would become available. Coil widths currently available are rather restrictive and only relatively narrow plates could be made in this way at present. However it is possible that available coil widths may increase and with coil widths of 2 metres or more folded plate elements well within the practical range could be made far more efficiently.

(b) *Sheets formed in a dishing press*

Conventional corrugated steel sheeting has further disadvantages when applied to folded plate work which may be readily overcome using the purpose-made form of sheeting illustrated in Figure 14. In Figure 14 the corrugations do not continue to edge of the sheet but are pressed out of the flat sheet in a manner more familiar in radiator panels. The corrugations can here run along the length of the coil or, more advantageously as in (a) above, run transverse to the length.

This arrangement has the advantage of almost completely eliminating the significant component $d_{1.1}$ of the deflection due to profile distortion and also avoids the problem with sheet/flange fasteners mentioned in section 4.3. Furthermore, weatherproofing at the fold lines is greatly simplified if fluted edges are avoided in this way.

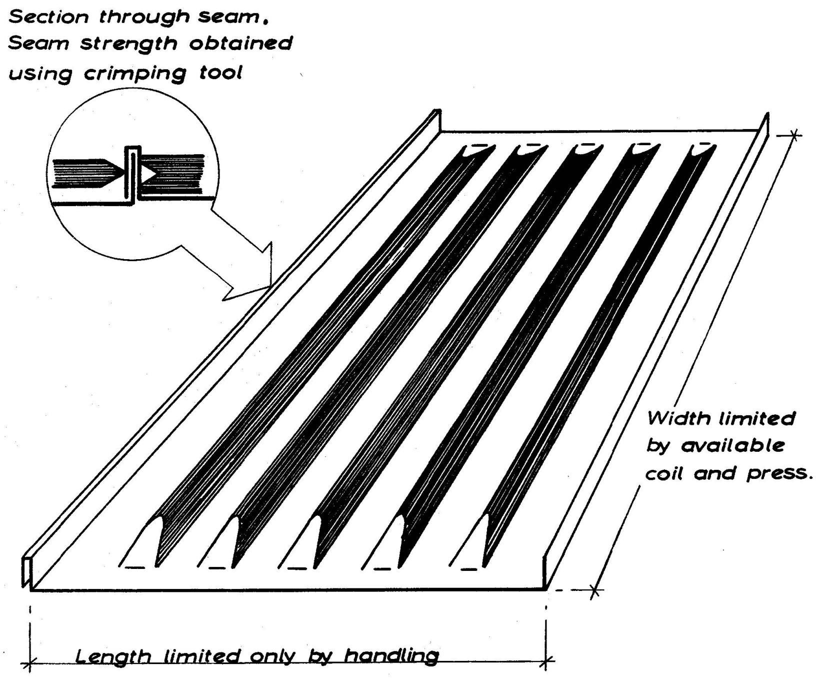

Fig. 14. Ideal Arrangement of Sheeting for Folded Plate Construction.

(c) *Alternative ways of forming seams*

The work reported in this paper has demonstrated that in the current state of the art fastening the seams between a large number of individual sheet widths may form a significant part of the fabrication process. Furthermore, self-tapping or self-drilling/self tapping screws have been found to be somewhat suspect in thin to thin connections where the individual sheet thicknesses are themselves at the thinner end of the practical range. This suggests that there may be better ways of making the seam connection than were used in the tests described previously particularly bearing in mind the necessity that the seams should be weather proof.

One possibility is also illustrated in Figure 14 where the seams are made weatherproof by forming a tight fold in one sheet covering an upstand in the adjacent sheet. Seam strength is obtained by crimping the three sheet thicknesses together using a specially designed tool. Other variations on this theme, which is being currently investigated at the University of Salford, are possible.

(d) *Sheeting running parallel to span*

The advantages of possibilities (a), (b) and (c) can be gained in an alternative way by providing light cross-members spanning between the fold lines and treating the sheeting as decking spanning over these cross-members. This requires a separate weatherproof membrane over the sheeting and it is a matter for the designer to decide whether the advantages outweigh the cost of the cross-members and weatherproofing. This possibility has been considered by THOMPSON [15] and will be discussed in more detail in a subsequent paper.

Control of Deflections

The deflections of folded plate roofs of span 15-20 metres or more are frequently significant. The dominant components of the total deflection are $d_{1.1}$ (distortion of the corrugation profile) and $d_{1.3}$ (axial deformation of flange members). Slip of fasteners is not usually of great significance. It follows that the most effective way to reduce deflection in an otherwise satisfactory design is to try to reduce $d_{1.1}$ either by fastening the ends of the sheets in every corrugation if this has not already been done or by using sheets formed in a dishing press as described in section 6(b). Alternatively the alternative arrangement mentioned in 6(d) may be considered.

Another effective way of reducing deflections is to increase the cross-sectional area of the fold-line members and this is often the main recourse when corrugation distortion has been minimised.

Safety and Economy

In order to be able to ensure the safety of this relatively novel form of construction it is necessary to be able to predict all the likely failure modes and to associate with each an accurate ultimate load. It is to this end that the tests described in this paper and in references 9 and 15 were carried out and it is now clear that the behaviour of individual folded plate elements is well understood. This test programme will conclude, probably in 1976, with the full scale test to destruction of a complete roof structure measuring 21.6 metres span by 10.8 metres wide. This test is under preparation at the University of Salford and it is considered that its successful completion will open the way to the much more extensive use of light gauge steel folded plate roofs by British and European designers.

The economic advantages of light steel folded plate construction are intuitively obvious but have not yet been subject to a rigorous investigation. One reason for this is the significance of the erection procedure and it is considered desirable to demonstrate the successful erection of a prototype structure before evaluating its economic advantages. Nevertheless it is already clear that over a wide range of spans a considerable saving in material is possible using light gauge folded plate construction as compared with alternative methods of covering the same span. It is also clear that single skin construction with discrete fasteners is considerably cheaper than the alternative welded configurations while, at the same time, possessing adequate stiffness.

Conclusions

Light gauge steel folded plate construction utilising a single skin of corrugated steel sheeting and discrete fasteners has been shown to be a viable method of construction for medium span roofs. The maximum spans possible will normally be limited by buckling and probably lie in the range 20-25 metres in the current state of the art.

There is scope for the development of alternatives to the utilisation of conventional roof sheeting and fasteners and some of these have been mentioned in the paper. It is clear that the efficiency of this form of construction will increase as further development takes place but it is considered that development has now reached the point where practical exploitation is possible and desirable.

Notation

Note. This notation is chosen to accord with that adopted by Bryan for shear panels (10).

A	cross-sectional area of flange member (715 mm^2).
b	depth of folded plate element (2540 mm).
d	pitch of corrugations (175 mm).
$d_{1.1}$, etc.	components of total deflection at centre of plate element (mm).
E	modulus of elasticity of steel (207 kN/mm^2).
F_p	ultimate load of individual sheet/flange fastener (3.83 kN).
F_s	ultimate load of individual seam faster (1.85 kN).
F_{sc}	ultimate load of individual sheet/gable fastener (3.83 kN).
g_1	factor to allow for number of sheet/flange member fasteners per sheet width (see table 2).
h	height of corrugations (35 mm).
K	constant for sheet distortion = 0.54 (p = 175 mm), 3.77 (p = 350 mm) or 8.01 (p = 525 mm).
L	span of folded plate element or roof (17600 mm).
n	number of seams in region of plate element between adjacent loads (see table 1).
n_f	number of sheet flange fasteners per flange per sheet width.
n_s	number of seam fasteners in each seam (see table 1).
n_{sc}	number of gable fasteners at each gable (see table 1).
p	pitch of sheet/flange fasteners (mm – see table 1).
r_1, etc.	lengths of panel associated with different fastener pitches (r_1 = 1760 mm, r_2 = 3520 mm, r_3 = 3520 mm).
s	slip per sheet/flange fastener (0.01 – 0.15 mm/kN).
s_s	slip per seam fastener (0.05 – 0.15 mm/kN).
s_{sc}	slip per gable fastener (0.01 – 0.15 mm/kN).
t	net steel thickness (0.66 mm).
W	jack load (working value 8.4 kN per jack).
v	poisson's ratio (0.25).

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Summary

This paper is concerned with the design of light gauge steel folded plate structures in which the plate elements comprise a single skin of corrugated steel shetting fastened together and to fold line members by discrete fasteners such as self tapping screws or blind rivets. An extensive test series on individual plate elements of 17.6 m span is described and the results compared with available theories. Various practical factors are also discussed.

Résumé

La contribution est consacrée à l'étude des toits plissés en éléments métalliques minces. Les versants sont formés d'une seule tôle formée à froid, la fixation entre les tôles elles-mêmes d'une part et avec les barres d'arête d'autre part étant réalisée par des vis autotaraudeuses ou par des rivets aveugles. Les auteurs décrivent une importante série d'essais portant sur des éléments de versant de 17,6 m de portée; ils comparent les résultats aux théories existantes. On discute également divers facteurs d'importance pratique.

Zusammenfassung

Die Autoren behandeln den Entwurf von Faltwerken aus Stahlleichtprofilen, bei denen die Scheibenelemente aus einer einzigen Lage profilierter Stahlbleche bestehen. Selbstschneidende Gewindeschrauben oder Blindniete dienen zur Befestigung sowohl der Bleche unter sich als auch der Bleche mit den Kantengliedern. Es wird eine ausgedehnte Versuchsreihe an einzelnen Scheibenelementen von 17,6 m Spannweite beschrieben, wobei die Ergebnisse mit vorhandenen Theorien verglichen werden. Ferner diskutieren die Autoren verschiedene Faktoren von praktischer Bedeutung.