Developments in Inexpensive Lightweight Structures

Michael J. Davies

DEVELOPMENTS IN INEXPENSIVE LIGHTWEIGHT STRUCTURES

by J Michael Davies* BSc PhD FICE FIStructE CEng

Introduction

The four structural systems described in this paper arose out of the desire to find new ways of using cold-formed light gauge steel in the manufacture of inexpensive low rise steel buildings. They represent notable examples of collaboration between University and industry in the development of new products.

1. A scheme for light gauge steel pitched roof construction

This structural system was devised with the large press brake capacity possessed by F E Titmuss (Constructional Engineers) Ltd very much in mind. It is a development of the familiar pitched-roof plane frame structure whereby the frames are cold-formed and all secondary members are eliminated. Apart from the gables, the complete structure has only two components namely the frames and profiled steel decking (or side walling).

The frames are pressed from flat sheet into an inverted hat section and are heavily tapered as shown in Fig. 1. The roof cladding has a 90 mm deep trapezoidal profile and spans directly between frames at 6 metres centres. A unique feature of the construction is the detail where the cladding passes over, and is connected to, the frames. As shown in Fig. 2, the cladding overlaps over the full width of each frame and is fixed through both sheet thicknesses to both flanges. This fixing is usually made by self-drilling, self-tapping screws of the 'Tek' type though there are various alternatives. The overall width of the section at the flanges

---

*Reader in Civil Engineering, University of Salford, England

is kept constant in order to provide fixing to the frame members at a uniform spacing regardless of variation in the depth of the frame members.

This form of construction possesses several notable advantages:

(a) The cladding is fixed in convenient span lengths of 6.6 metres and the fixing detail ensures that it behaves almost as though fully continuous as far as both strength and stiffness are concerned.

(b) There are no secondary members so that erection is both simple and fast.

(c) Although the frame members are cold-formed from light gauge steel, there are no problems of lateral or torsional instability. The system is self-bracing.

(d) The structure is attractive in appearance with particularly clean internal lines and contains no dust traps.

(e) It can be economically made in a range of spans from a stockpile of a small number of coil sizes.

Fig. 3 shows a view of the interior of a section of structure erected for test purposes. It may be noted that in addition to conventionally profiled roof cladding, a side cladding was developed with an unsymmetrical profile designed to shed water. Thus the roof requires an external weatherproof surface but this is not necessary for the sides.

As this was a completely novel structural conception, it was decided that it should be verified by testing by the Civil Engineering Department of the University of Salford. The initial brief called for four alternative frame sizes with spans of

12 and 15 metres and heights to eaves of 3 and 5 metres. The smallest of these was chosen for testing purposes having a span of 12 metres and a height to eaves of 3 metres. The testing of the complete concept took place in three distinct phases.

(a) Testing of the sheeting and fixing details

These tests were conducted in order to verify that the roof cladding, and in particular its fixing detail, was capable of the assumed performance. It was necessary to show that the lengths of decking were capable of carrying load in the fully fixed condition up to the design load without inducing either web crippling or premature failure at the fasteners. The latter factor is crucial to the conception and is not amenable to rigorous analysis for either serviceability or strength.

The dimensions of the profile used for the tests are shown in Fig. 4. This profile was designed to the requirements of the Metal Roof Deck Association(1). The critical support bending moment was calculated assuming the sheeting to be fully continuous over twin supports at 0.45 m centres. The buckling strength of the web was checked according to the requirements of the A.I.S.I(2).

The decking was tested over three unequal spans as shown in Figs. 5 and 6 in order to reproduce typical behaviour in the centre span without premature failure of the outer spans which had simple supports at their outer ends. The central span of decking was fastened to twin R.S.J. supports at 450 mm centres with a single 6.1 mm dia. Tek fastener through each corrugation trough reproducing the detail shown in Fig. 2.

Two separate tests were carried out. In the first test, only the centre span was loaded and the central deflection measured as the load was increased in increments up to failure. The deflection at the working load was span/353 which was greater than the fully fixed value of span/553 but considerably less than the usually adopted value of span/184 (calculated as $\Delta = 3wL^4/384\ EI$). The load causing failure in bending was 2.25 kN/m^2 which was lower than the expected value of about 2.45 kN/m^2. However, the gross metal thickness of the galvanised sheets used for the tests was found to be only 0.615 mm giving an estimated net steel thickness of the order of 0.56 mm and, bearing this in mind, the results of the test were considered to be satisfactory. The yield stress was measured as 310 N/mm^2 based on the estimated net steel thickness.

For the second test all three spans were loaded, the test being similar in all other respects to the first. This proved to be a more severe loading condition than the first and the deflection at the working load was increased to span/290. The test was terminated at a load of 1.94 kN/m^2 with evidence of bending failure both at mid-span and at the supports although the sheeting was still supporting the load.

In neither test was there any evidence of significant distortion at the lapped fixing despite the comparatively rigid detail provided in the laboratory. The sheeting was therefore found to be adequate in respect of both serviceability and strength.

(b) Testing of the complete structure-phase 1

For this phase of the testing, three complete bays of a structure of 12 m span and 3 m height to eaves were erected on a prepared outdoor test side immediately adjacent to the Structures Laboratory at the University of Salford, as shown in Fig. 3. The erection was carried out extremely rapidly and without difficulty by a two man erection team experienced in the erection of light steel buildings though of course new to this particular configuration.

The testing was carried out in two phases, the first involving loading the two centre frames through the sheeting. A sophisticated system was used in which load was generated by four hydraulic jacks and applied to the structure through a grillage system as shown in Fig. 7. This arrangement gave two line loads on the roof as shown in Fig. 8 and was designed to produce the correct relationship between bending moment in the sheeting and load on the two inner frames.

As expected this test led to failure of the sheeting without any visible sign of failure in the frames. In fact the sheeting failed prematurely in this test as compared with the laboratory test at a load of 1.50 kN/m^2 but failure was seen to be initiated at the apex where a detail had been adopted which was not completely satisfactory. This failure was evidently aggravated by the use of interconnected line loads in lieu of distributed loads. The mode of failure is shown in Fig. 9. As time was important and as the performance of the sheeting in the laboratory test had been satisfactory it was not considered necessary to repeat the test.

During testing, deflections were measured at each load increment at a number of points on the structure and the results can only be summarised here. The sheeting deflection was measured at the mid point of each of the line loads and the corresponding deflections of the frames were also read. The difference between the two gave a deflection of about 18 mm for an imposed load equivalent to 1.0 kN/m^2. The corresponding theoretical value is difficult to establish with precision as it is dependent on the flexibility of the frames at the point of measurement but is certainly close to this value.

The points of attachment of the sheeting to the frames were again examined during the test and, as in the previous laboratory tests, no serviceability problems were observed prior to failure.

The deflections of the frame proved to be somewhat smaller than anticipated. The apex deflection was measured as approximately 31 mm at the working load whereas the expected values were 41 mm for a pinned-based frame and 30 mm for a fixed base frame. These values were obtained using a computer program for plane frame analysis in which each tapered member was replaced by eight short lengths of uniform section. The analysis may have been influenced by the use of nominal dimensions with the consequent neglect of the considerable depth at the haunch. As the bases were flat plates with a single bolt to either side, some degree of base fixity may have contributed to the reduction in deflection. Continuity of bending action in the deeply profiled sheeting may also have shed some load to the less heavily loaded outer frames.

(c) Testing of the complete structure - phase 2

As phase 1 of the test program had given only limited information regarding the strength of the frames a further test was carried out whereby load was applied direct to a single frame as shown in Fig. 10. As it was suspected that the previous test had been influenced by some degree of base fixity an effort was made to reduce this for the final test. The bases were raised and a 12 mm diameter rod was introduced as near to the centre line as possible before the holding down bolts were retightened.

Load on the frame was increased in increments up to failure and deflection measurements taken on both the loaded frame and the adjacent frames in order that any load shedding through the sheeting could be determined and allowed for. The salient results from the test are summarised below:-

Design load per frame	= 82.8 kN
Failure load	= 184 kN
Failure load allowing for load shedding	> 150 kN
Load factor at failure	> 1.81
Measured yield stress of material	= 237–257 N/mm^2
Predicted apex deflection at working load	= 41 mm
Measured apex deflection at working load	= 26 mm
Measured value allowing for load shedding	= 34 mm

Clearly the behaviour of the frame was in all respects satisfactory. Since developing the frame system described above, the basic conception has also been applied to flat roof construction in which the same sheeting detail is used in conjunction with pressed steel beams of constant depth spanning over conventional columns. A computer program has been developed to optimise the cross section. Simple spans or spans with cantilever overhangs can be designed.

2. Stressed skin monopitch construction

A typical stressed-skin monopitch structure is shown in Fig. 12. In this innovation the intention is again to reduce the number of components in the structure by utilising relatively deeply profiled steel cladding and eliminating secondary members. The basic conception relies on the use of the roof sheeting both as a primary member spanning the full width of the building and also as a stressed skin girder. Thus the side loads on the structure are all carried back to the gables in the plane of the roof and through the gable cladding to the foundations. There are a number of variations possible on this basic theme.

The posts shown in Fig. 12 carry vertical load only and indeed these can in many instances be replaced by cladding spanning vertically carrying a combination of bending and in-plane forces. In this structure all of the components are light and easily handled and only simple fasteners (Tek screws, pop rivets or nuts and bolts) are used. Technically, the main problem concerns the design of the roof.

Design of the roof

The roof must be designed to fulfil two separate functions. In the first place the roof sheeting must be designed for its primary purpose of carrying imposed vertical and normal loads back to the flange members in simply supported bending. For this part of the design it is merely necessary to consult manufacturers safe load tables, or to adopt conventional calculations.

The roof must then be designed as a stressed skin girder to span between the gables of the structure. In performing this function it acts in a manner very similar to a deep plate girder in which the 'flanges' carry the bending moment in axial tension and compression and the sheeting acts as the 'web' carrying the shear force. However, the problem is somewhat more complex than is indicated by this simple analogy and the detailed theory and its justification will be found in references 3-6. The dating of these references indicates the recent nature of the development work and perhaps explains why the essentially very simple concept shown in Fig. 12 has not found earlier implementation.

It should be pointed out that the above references refer to the development of light gauge steel folded plate construction. However, the basic design problem with this form of folded plate structure is to predict the behaviour of an individual plate and to this end a great many tests have been carried out at the University of Salford on individual plate elements of spans up to 21.6 metres. Fig. 13 shows one such test in which the span was 17.6 metres and the depth 2.54 metres and it can be seen that the test arrangement reflects precisely the structural form and the type of loading found in a stressed skin monopitch structure. Over a wide range of situations it has been shown that the test values of both strength and stiffness can be readily predicted using the theories developed in references 3-6.

The possibility of stressed skin monopitch construction arises largely as a consequence of the availability of deep profiled steel sheeting. The theoretical treatment described above embraces the deeper profiles and associated widths of construction without modification and it was therefore not considered necessary to perform any additional tests in order to validify this form of construction.

The stressed skin monopitch is an inherently strong structure and in most instances the maximum length between gables will be limited by horizontal deflection. This deflection arises mainly as a consequence of shear distortion of the corrugated profile of the sheeting which current theories can predict with considerable accuracy[6]. There is no problem in obtaining adequate strength and frequently nominal fastener spacings will be sufficient.

Other Design Considerations

As has already been pointed out, the lateral load is transmitted from the roof to the foundations through the gables. This may involve conventional diagonal bracing but preferably the design of the gable may invoke stressed skin principles and utilise the in-plane strength of the gable sheeting. The gable then becomes a simple diaphragm or shear panel for which well known principles exist. Reference 7 provides the background and reference 8 the up-to-date design procedure.

There are two other points of note in this unconventional design. Firstly, the small loads due to wind forces along the length of the building will normally be readily accommodated by stressed skin action in the cladding of the long sides. Secondly the longitudinal members forming the flanges of the roof girder must be continuous with respect to the rather small axial forces induced by stressed skin action and must also be capable of spanning between points of support under the vertical loads arising from their function as lines of support for the roof sheeting.

3. "Floclad" Mansard Units

The primary structure of the 'Floclad' mansard unit is entirely frameless consisting only of profiled steel sheeting with local curved regions formed by a simple pressing process. In this process the sheeting is deliberately crimped at intervals, the radius of curvature being determined by the frequency of the crimps. In the mansard units, the sheeting is curved in three regions, as shown in Fig. 14, to form a structure which behaves as a continuous arch with no prismatic members necessary other than those required to form the gables.

The primary problem here is the determination of the relevant structural properties of the curved regions of sheeting and a test to determine the strength and stiffness in hogging bending is shown in Fig. 15. Tests to determine the properties in sagging bending were carried out in a similar fashion with the load reversed.

The profile used for this investigation was a conventional trapezoidal section of 38 mm depth and with nominal thicknesses of 0.7 and 0.9 mm. Sheets of this profile were formed into semicircles with initial radii in the range 380 mm to 900 mm and tested to determine their flexural regidity and the ultimate amount of resistance.

The moment of resistance was found to be independent of the radius of curvature though different from that of the undeformed sheets and strongly influenced by the direction of loading as shown in Table 1. The flexural rigidity was found to be dependent on the radius of curvature as shown in Fig. 16.

Thickness (mm)	Direction of loading	Ultimate moment (kNm/m)
0.7 mm (undeformed)	hogging sagging	1.84 2.10
0.7 mm (deformed)	hogging sagging	0.73 2.26
0.9 mm (undeformed)	hogging sagging	not tested
0.9 mm (deformed)	hogging sagging	1.12 3.45

(note - hogging implies wide flange in compression)

Table 1 Ultimate moment of resistance of floclad sheets

With the aid of the material properties found as a result of the above tests, a number of trial designs were analysed using a conventional plane frame matrix displacement method of analysis in which the curved regions were replaced by a small number of prismatic members. Several feasible designs were obtained of which a typical example is shown in Fig. 17. The size of this frame is such as to make it commercially viable and it is hoped to carry out a full scale test in the near future. The profile depth for the frame shown is 38 mm with a nominal material thickness of 0.9 mm. The calculated apex deflection is 57 mm under a uniformly distributed load of 0.9 kN/m^2 and the anticipated load factor against collapse is of the order of 2.4 with first yield taking place in the curved regions at the eaves at a load factor of about 1.5. It should be noted in this respect that the eaves regions are in hogging bending and have relatively low strength but high ductility and considerable redistribution of force may confidently be anticipated.

4. Light gauge steel folded plate roofs in the U. K.

Basic development work on light gauge steel folded plate roofs in the U. K. concluded with the publication of two papers describing the basic theory[3] and the full scale testing of a 21.6 m span structure[5]. A hipped plate roof has also been subjected to onerous acceptance tests[9].

This development work has been concentrated on roofs fastened entirely with mechanical fasteneings (self-drilling, self-tapping screws; blind rivets; fired pins etc). Both conventional trapezoidally profiled sheets and purpose made sheets have been considered. The final test to failure of the 21.6 m span roof shown in Fig. 18 attracted a large audience and the resulting publicity has done much to promote this form of construction.

Preparations to build the first permanent structures to the above principles are at an advanced stage.

Conclusions

Four novel forms of light weight construction have been described and their development into practical structures has been discussed. In each case it has been found that significant economies can be made in comparison with alternative ways of manufacturing similar buildings. These economies arise not only as a consequence of using material more efficiently but also as a consequence of greatly simplified detailing and erection.

Acknowledgement

The author wishes to thank F E Titmuss (Constructional Engineers) Ltd for permission to describe the development work on both the light gauge steel pitched roof construction and stressed skin monopitch construction and Ash and Lacy Steel Products Ltd for permission to describe the work on 'Floclad' Mansard Units. The work on light gauge steel folded plate roofs has been supported by the British Steel Corporation.

This development work has all taken place at the University of Salford and the help and encouragement of Professor E R Bryan is greatly appreciated both in the projects themselves and in the collaboration between University and industry that made them possible. The full scale testing described in the paper would not have been possible without the enthusiastic cooperation of many members of the Civil Engineering Department. The contribution of Mr W Deakin and Mr J Grimshaw is particularly acknowledged.

References

1. "Code of design and technical requirements for light gauge metal roof decks" Metal Roof Deck Association, London 1965

2. Specification for the design of cold-formed steel structural members" American Iron and Steel Institute Washington 1968

3. J M Davies "Light gauge steel folded plate roofs" Structural Engineer Vol 54 No 5 May 1976

4. J M Davies and F Thompson "Light gauge steel folded plate construction" IABSE Memoires Vol 36 - II 1976

5. J M Davies, E R Bryan and R M Lawson "Design and testing of a light gauge steel folded plate roof" Proc. I.C.E. Part 2 Vol 63 June 1977

6. J M Davies and R M Lawson "The shear flexibility of corrugated steel sheeting" Proc. 3rd Int. Spec. Conf. on Cold-formed steel structures. Univ. of Rolla-Missouri Nov. 1975

7. E R Bryan "The stressed skin design of steel buildings" Crosby Lockwood Staples, London 1973

8. J M Davies "Calculation of steel diaphragm behaviour" J.Struct.Div. Proc. ASCE, Vol 102 No ST7 July 1976

9. J M Davies, R M Lawson and J G Young "Calculated and observed behaviour of the folded plate roof of a nursery school" Proc. Int. Conf. on the Performance of Building Structs. Glasgow 1976. Pentech Press, London 1976

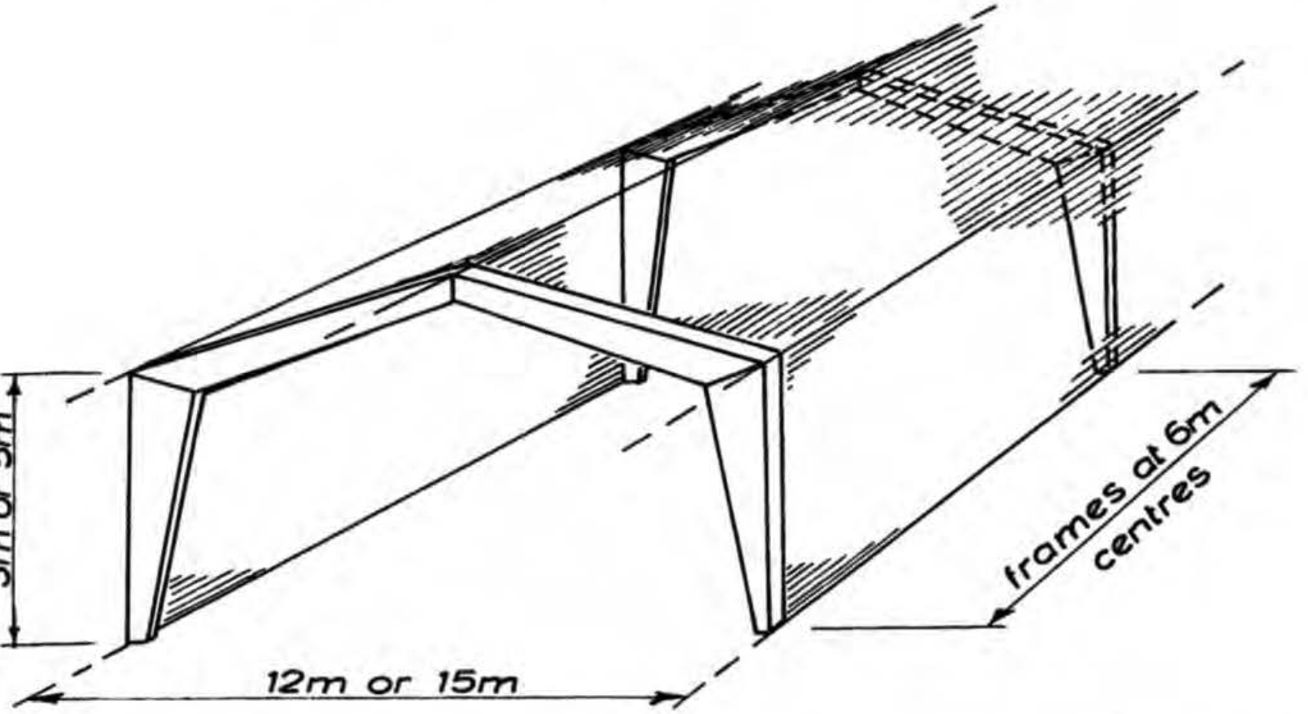

Fig 1 General arrangement for light gauge steel framed construction.

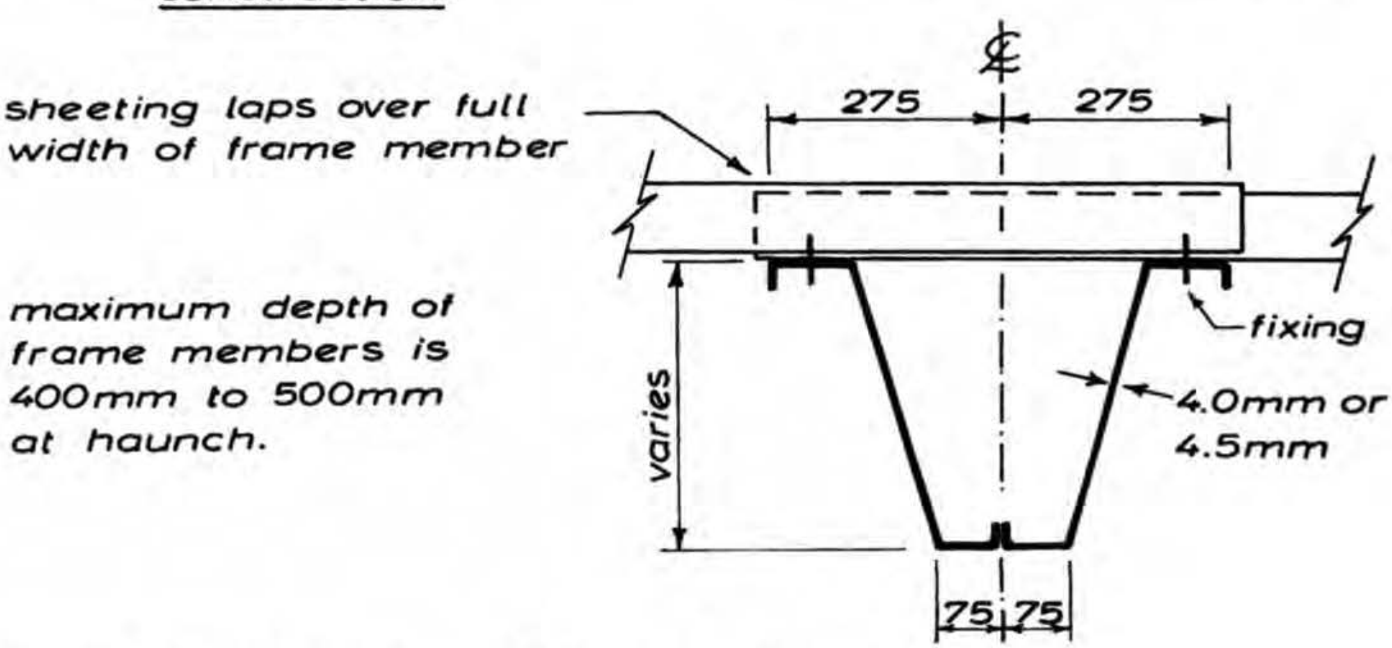

Fig 2 Cross section of frame members.

Fig 3 Section of structure erected for testing.

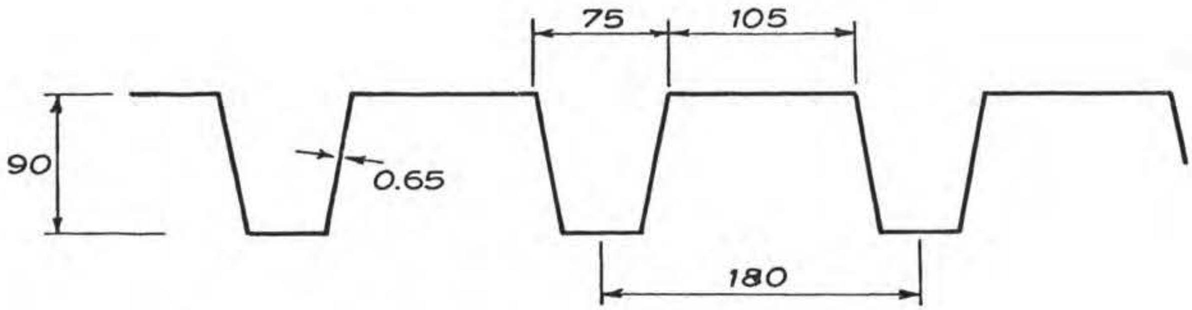

Fig 4 Profile of roof cladding. (dimensions in mm)

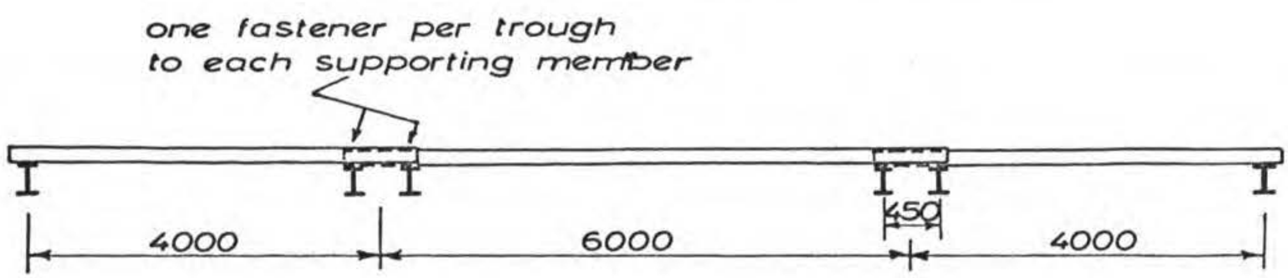

Fig 5 General arrangement for testing roof cladding.

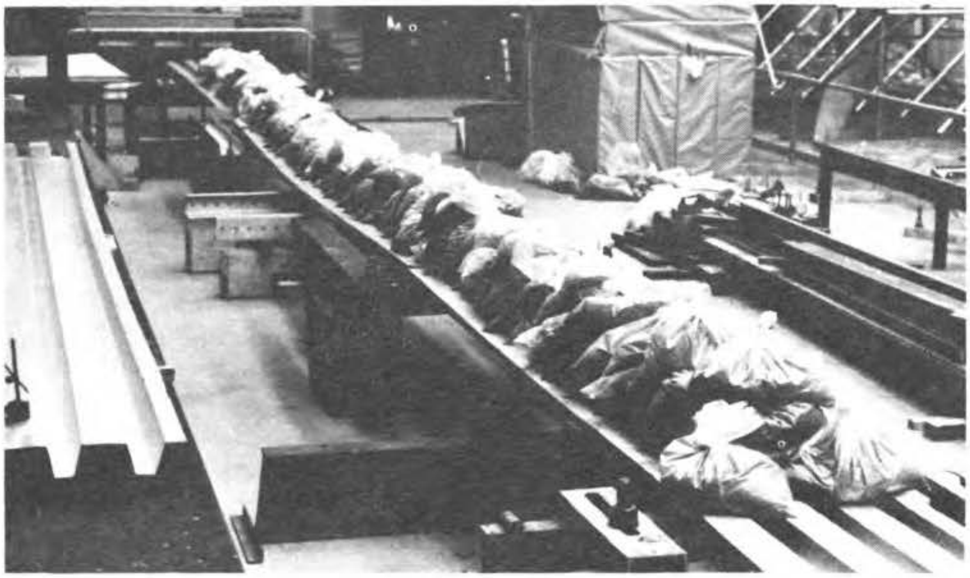

Fig 6 Roof cladding under test.

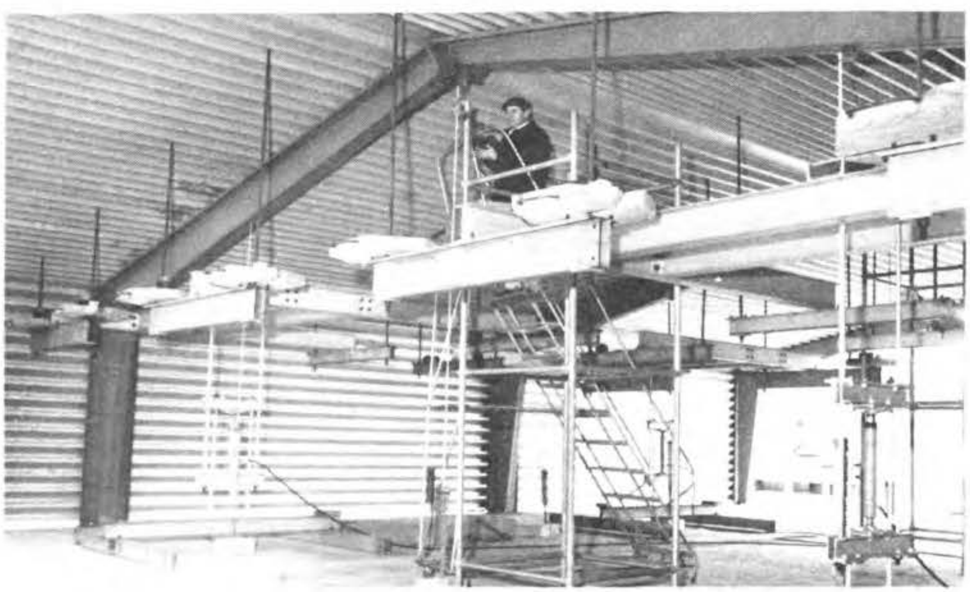

Fig 7 Interior of test structure showing loading system.

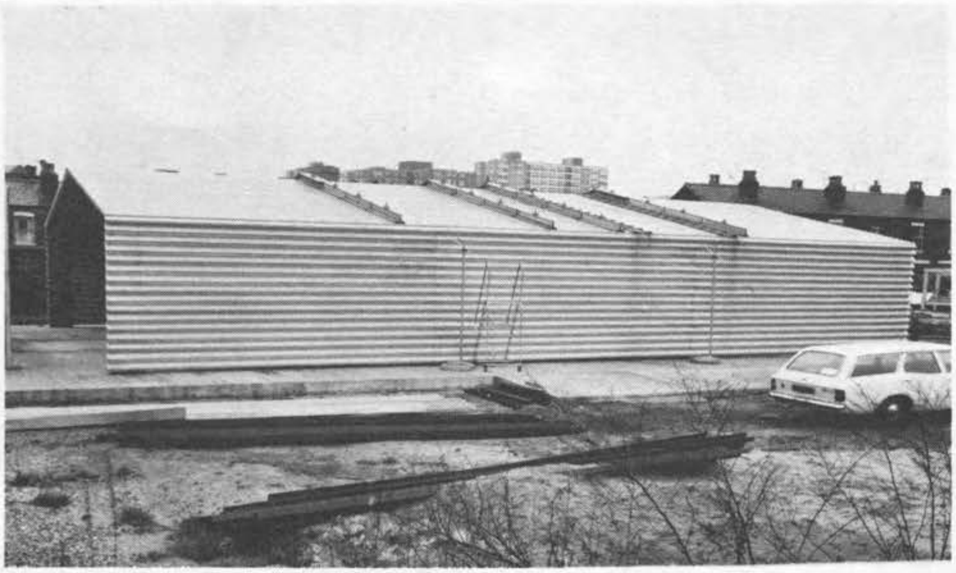

Fig 8 Exterior view during testing.

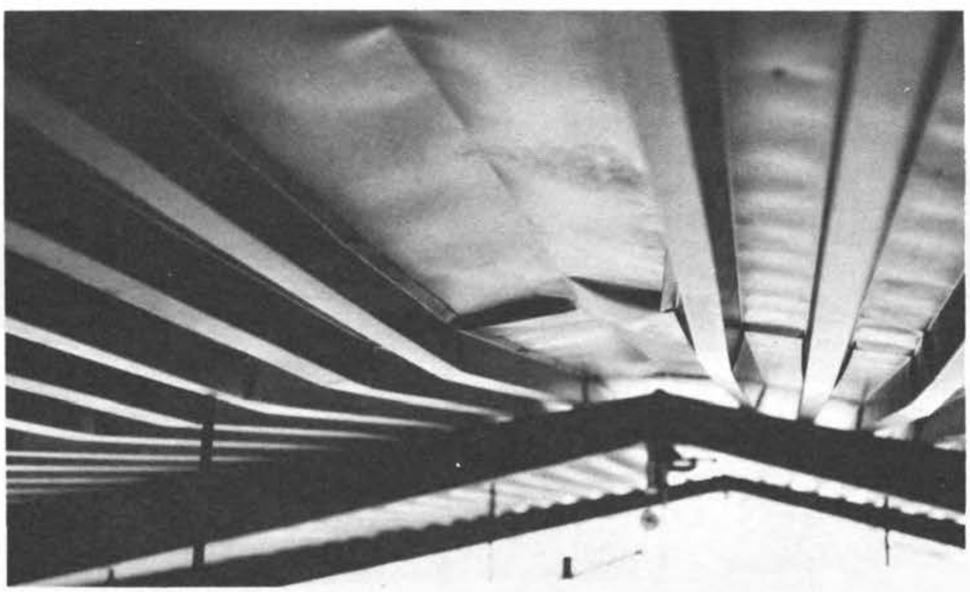

Fig 9 Failure of sheeting at apex.

Fig 10 Load applied direct to a single frame.

Fig 11 Eaves after failure.

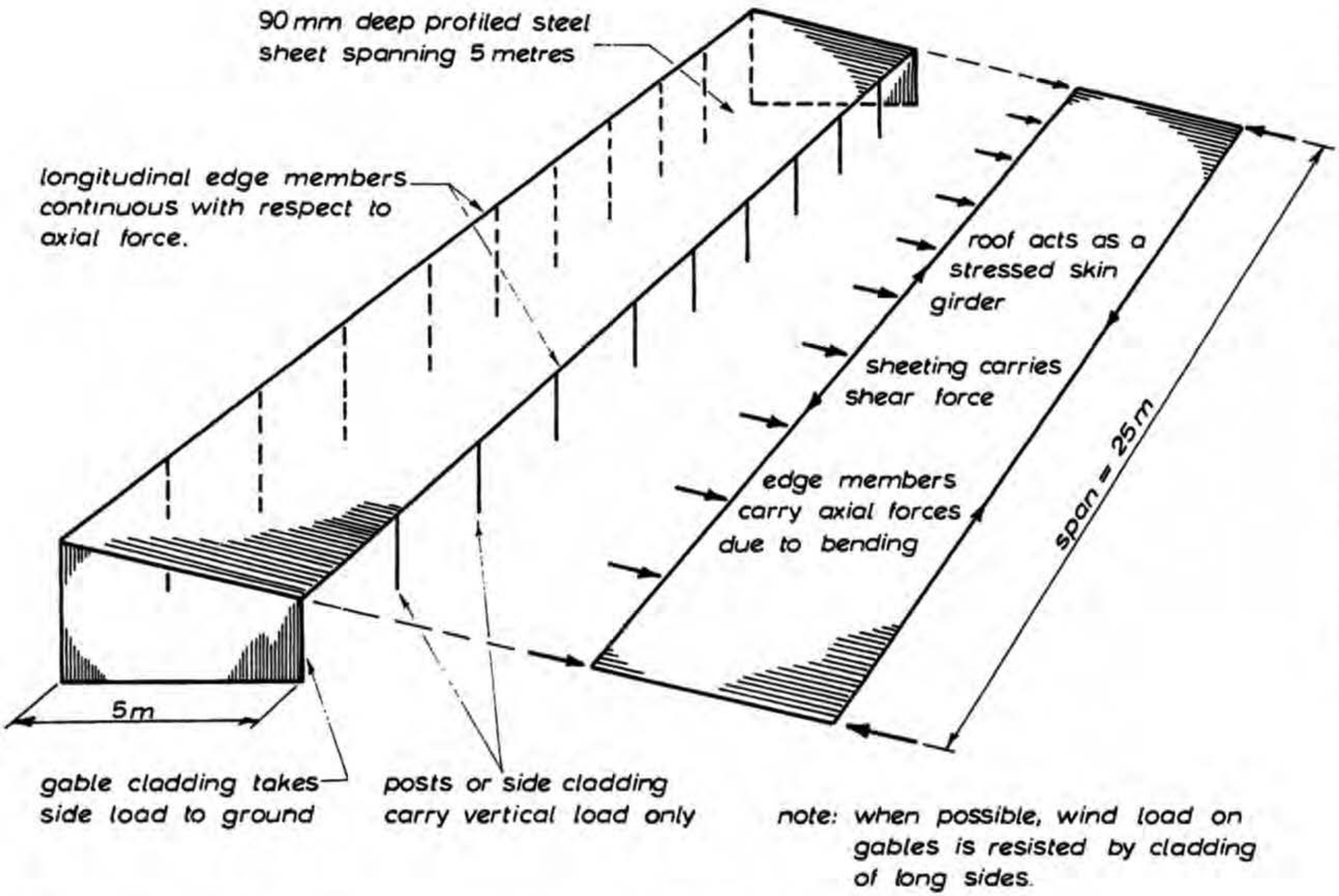

Fig 12 Scheme for a stressed skin monopitch structure.

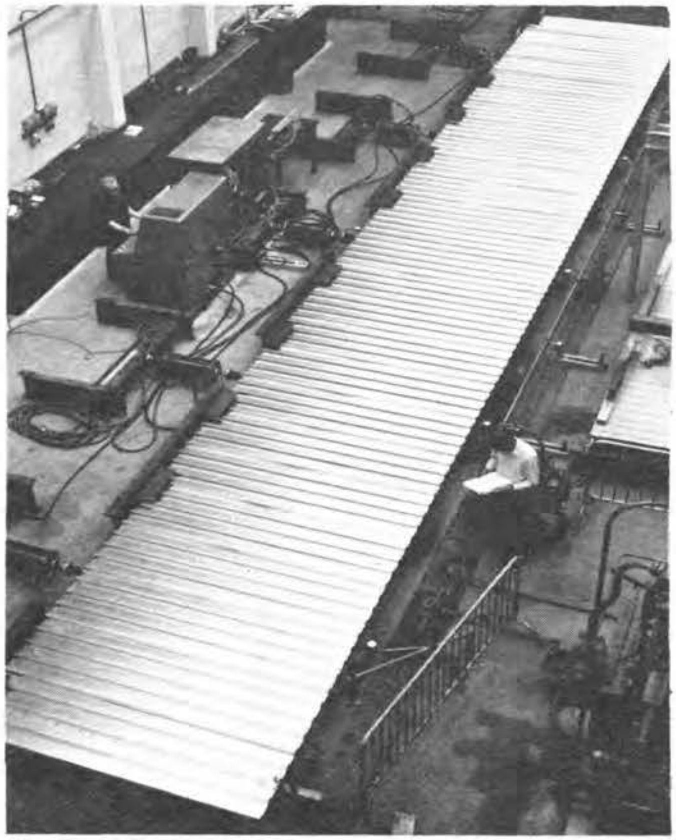

Fig 13 Stressed skin girder under test.

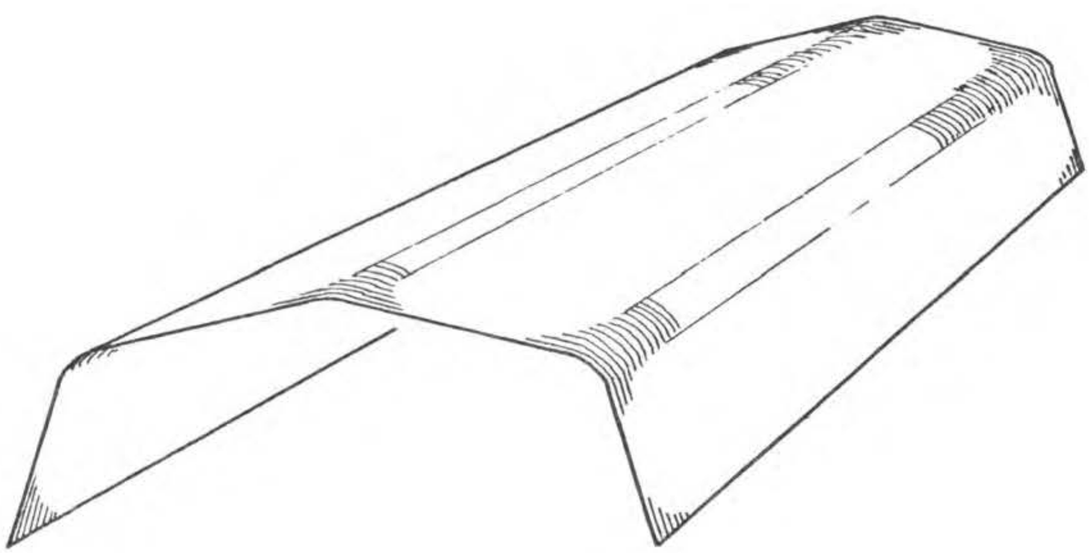

Fig 14 Scheme for 'Floclad' mansard units.

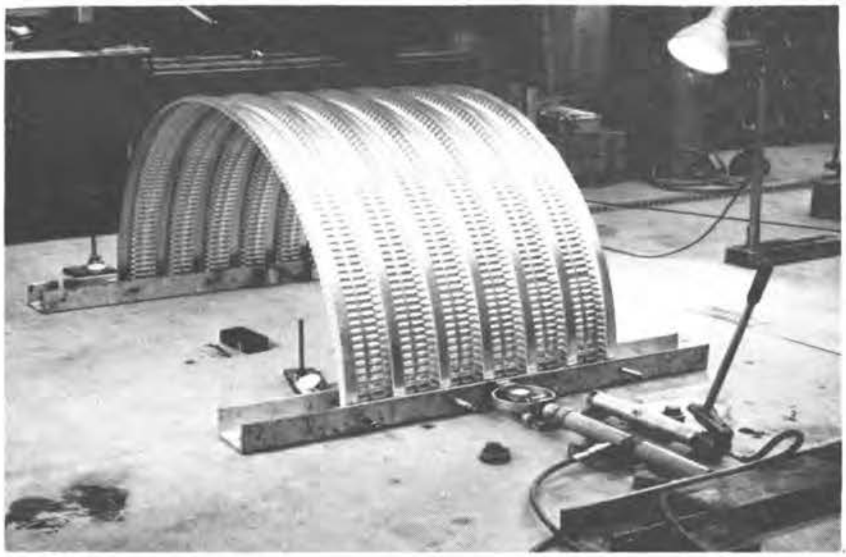

Fig 15 Determination of properties of 'Floclad'.

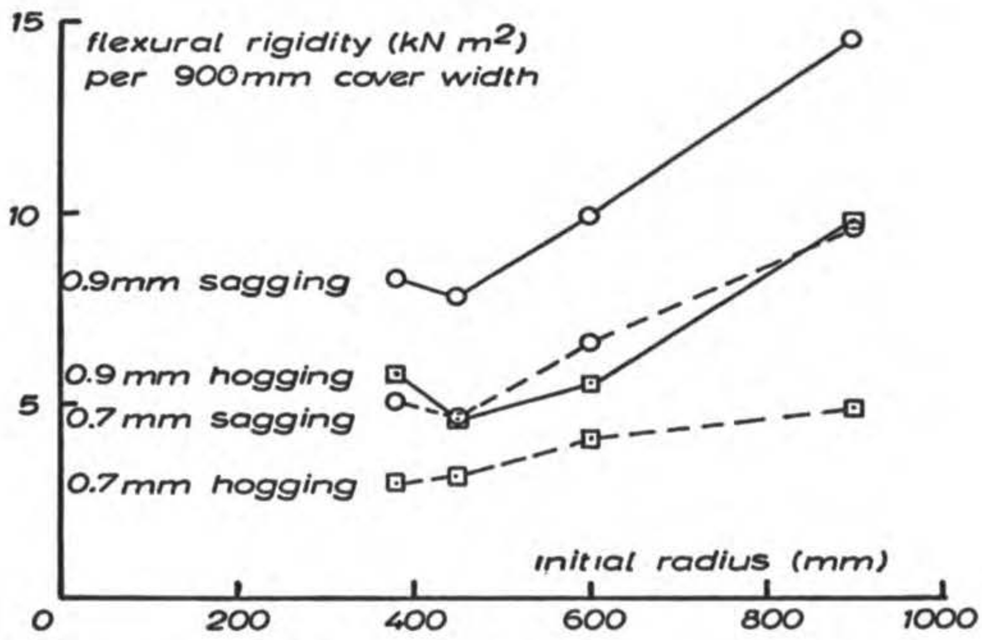

Fig 16 Variation of flexural rigidity of 'Floclad' with radius.

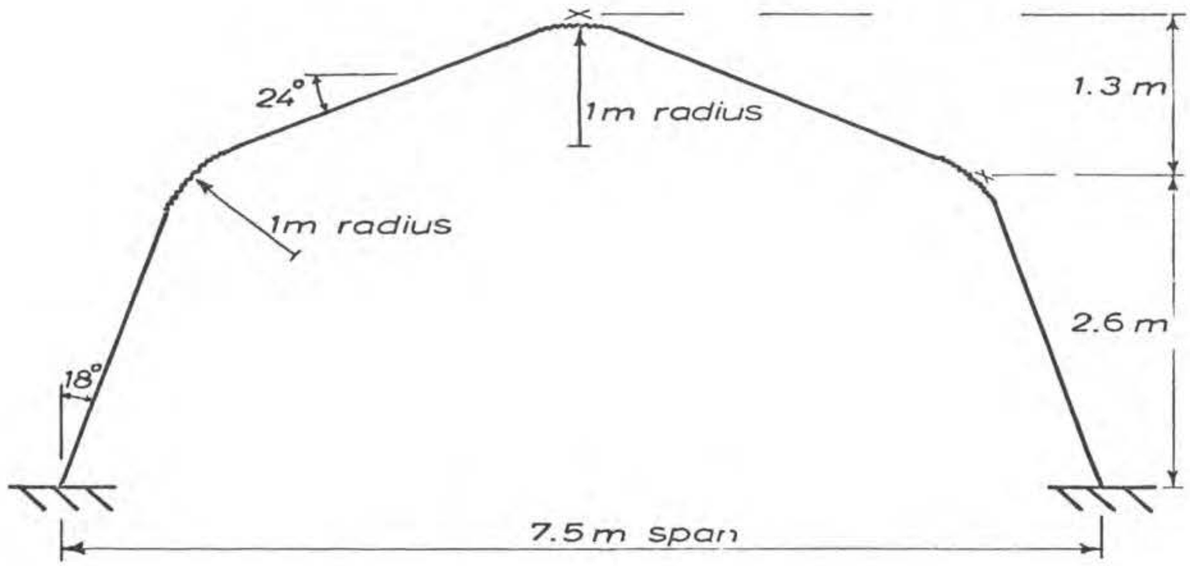

Fig 17 Cross section of typical fixed base mansard unit.

Fig 18 Full scale folded plate roof under test.