Full scale tests and analysis of two light gauge steel pyramid roofs

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Full Scale Tests and Analysis of Two Light Gauge Steel Pyramid Roofs

Essais et analyse de toitures pyramidales tronquées en acier formé à froid

Bauteilversuche und Berechnungen von zwei Pyramidendächern in Leichtbauweise

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SUMMARY

This paper reports on tests on a single hipped roof structure and a two-unit prefabricated
development. At the time of testing, it was not found possible to provide an analysis for the
asymmetrical load case which relies for its stability on stressed skin (diaphragm) action. In this
paper, a suitable analysis is developed and a comparison between the experimental and
theoretical deflections is made. Consideration is also given to the prediction of the forces in the
fasteners connecting the sheeting to the framing members.

RÉSUMÉ

Cet article rapporte les essais effectués sur des structures pyramidales tronquées uniques ainsi
que sur celles composées de deux unités préfabriquées. Au moment où les essais ont été
effectués, il était impossible de fournir une analyse pour un cas de chargement asymétrique qui
mette à contribution pour sa stabilité l'effet de diaphragme. Dans cet article une analyse
spécifique est développée et une comparaison entre flèches expérimentales et théoriques est
effectuée. On fait également des considérations quant à la prévision des efforts dans les fixations
liant la couverture à l'ossature.

ZUSAMMENFASSUNG

Es wird über Versuche an einem einfachen und einem doppelten Pyramidendach berichtet, für die
eine Vorfertigung entwickelt wurde. Zur Zeit der Versuche war eine rechnerische Behandlung des
unsymmetrischen Lastfalls, bei dem die Tragfähigkeit auf der Scheibenwirkung beruht, noch nicht
möglich. In diesem Beitrag wird eine geeignete Berechnungsmethode entwickelt. Vergleiche
zwischen rechnerisch und experimentell ermittelten Durchbiegungen werden vorgenommen.
Ausserdem werden die Kräfte in den Befestigungen zwischen den Blechen und den Trag-
elementen betrachtet.

1.   INTRODUCTION

1.1   The MACE Structure

The 'MACE' Structure was originally conceived by the Metropolitan Architects Consortium for Education as a low cost unit designed to meet the needs of nursery education. The novel feature of the structure is a roof with the shape of a truncated pyramid as shown in Fig. 1. Light framing members form the edges of four trapezoidal folded plates and these comprise the complete roof structure which relies for its stability on diaphragm action in the profiled steel cladding. At the time these units were designed, it was not possible to offer a satisfactory analysis of the behaviour of the structure under asymmetric loading and it was deemed necessary to subject a typical unit to acceptance tests to BS 449 [1]. Fig. 1 shows the dimensions of the tested structure and Fig. 2 shows the structure under test.
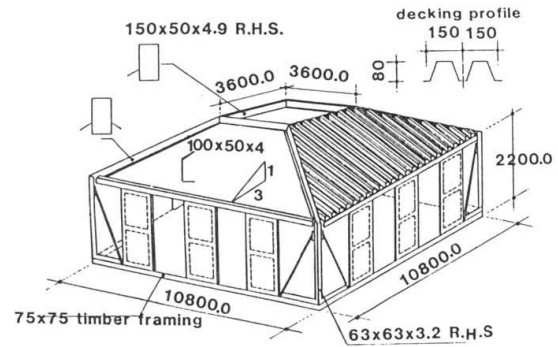
Fig. 1   MACE Type 30 unit as tested.

The roof sheeting had a trapezoidal profile with a depth of 80mm and a net steel thickness of 0.67mm. It was fastened to the supporting members with 6.1mm diameter self-drilling, self-tapping screws through the troughs of alternate corrugations. The seam fasteners were self-drilling, self-tapping screws of 4.1mm diameter at 250mm centres. More complete details of the structure and tests are given in [2] and [3].

Fig. 2   MACE unit under test   Fig. 3   Pyradomes in use

1.2   The Pyradome

The 'Pyradome' is a development of the MACE concept in which the individual trapezoidal panels are prefabricated for rapid erection and in which there is a facility for eaves members to span 10 metres between corner columns. A number of structures have been built using assemblies of Pyradomes as shown in Fig. 3. During development, a two module assembly was tested at the University of Salford. Fig. 4 shows the general arrangement and Fig. 5 the structure under test.

The members of the Pyradome roof comprise an upper compression ring which supports a plastic roof light and a lower tension ring at eaves level, the two being separated by sloping hip members. Steel cladding, spanning between the eaves and upper ring members, serves both to carry asymmetrical load by diaphragm action and to contribute to folded plate behaviour when adjacent modules act together, thus permitting a clear internal span between corner columns. Edge members required either an intermediate column or a

strengthening member as shown in Fig. 4. However, subsequent development has resulted in the use of an eaves beam which can clear span 10 metres.
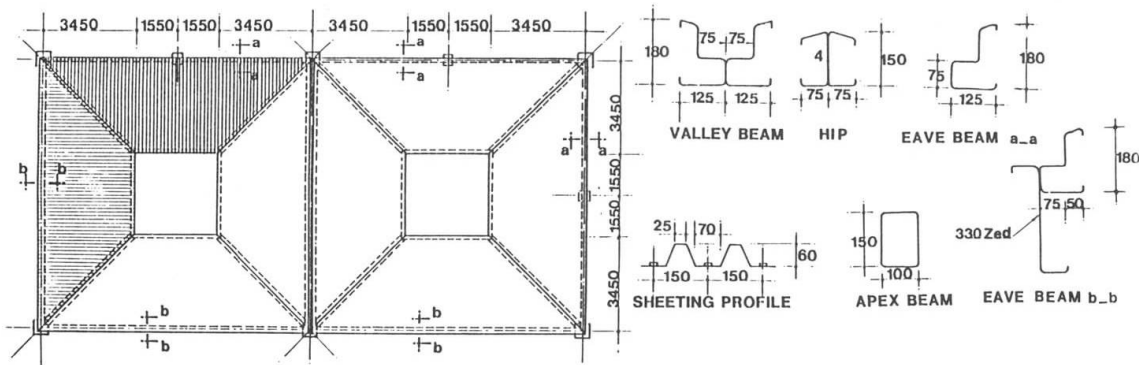

Fig. 4   Plan of tested Pyradome structure

The trapezoidally profiled steel cladding had a depth of 60mm with a nominal thickness of 0.7mm and, in contrast to the MACE panel, was fastened to the supporting members in every trough using 6.1mm diameter self-drilling, self-tapping screws. Seam fasteners were also self-drilling, self-tapping screws at 200mm centres.

The test load levels were based on the draft revision of BS 449 which was current at the time of testing (1980). Based on a working load of $1.0kN/m^2$ and a dead load of $0.27kN/m^2$, the loading sequence was as follows:

> Test 1: Load in five increments up to working load on one Pyradome followed by similar loading on the other.
>
> Test 2: Load up to acceptance load ($1.39kN/m^2$) on both Pyradomes in increments.
>
> Test 3: Load the whole area of the roof up to half the working load.
>
> Increase the load on half of the area (shown shaded in Fig. 13) to full working load. Apply uniform working load to the whole roof and increase in increments up to the prototype test load of $1.68kN/m^2$.

Loading was applied using 25kg sand bags. At each load increment and after the test loads had been maintained for 15 minutes, the deflections at various points were recorded. Vertical deflections were measured by observing suspended scales through an optical level and lateral movements were measured by theodolite. After each test the load was reduced to the factored dead load and the residual deflections recorded.
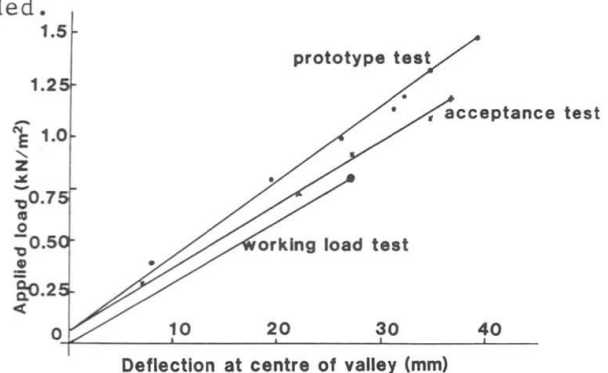

Fig. 5   Pyradome under test

Fig. 6   Typical load-deflection curves for Pyradome.

1.3 Test results and previous analyses

From the practical point of view both the MACE and Pyradome tests were seen as non-destructive tests of prototype structures. The success of the test was assessed in terms of the percentage recovery of deflection and no calculations were required.

For the MACE structure, detailed test results and recovery values are given in [2]. Some simple calculations were carried out for the uniform load case, where stressed skin action has secondary importance, and preliminary consideration was given to the forces arising as a consequence of diaphragm action using finite element analysis of individual trapezoidal panels. However, no calculations were attempted for the assymmetrical load case.

For the Pyradome structure, no calculations had been undertaken prior to the present work. A typical set of load deflection curves is shown in Fig. 6. From these and other similar graphs, it can be concluded that the response is linear and for subsequent comparisons, it will be sufficient to consider patternsof deflection at a given load level. These will be given later after the theoretical analysis has been discussed.

2.   MATHEMATICAL MODELLING

2.1 Models for individual panels

Two-dimensional analysis of the individual trapezoidal roof panels using finite elements had been carried out previously for the MACE structure [2]. The mathematical model used, which included detailed consideration of individual fasteners, is shown in Fig. 7. There are considerable practical difficulties in extending this model to the full three-dimensional roof structure.
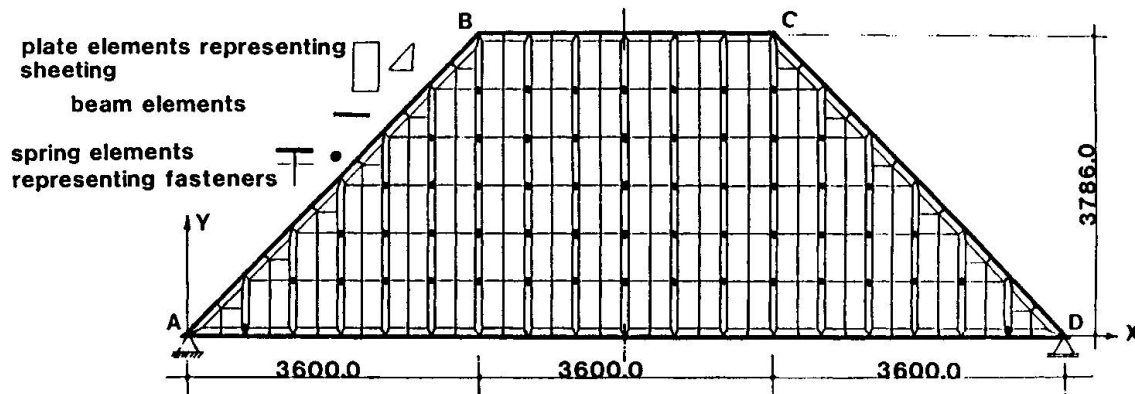

Fig. 7   Finite element model of MACE roof panel.

However, it has been shown [6] that the detailed behaviour of both regular and irregular diaphragms can be accurately simulated using a plane truss model. The original concept requires modification to deal with the trapezoidal shape and a suitable representation is shown in Fig. 8.

This model includes additional 'spring' members to simulate the flexibility of the sheet to edge member fasteners in a direction parallel to the edge members and can be used to analyse a complete hipped roof structure using a suitable computer program for the analysis of space structures. This is, however, a very cumbersome operation and, for practical purposes, it is necessary to seek further simplification.

1. Beam element
2. Shear stiffness of sheeting
3. Stiff vertical member
4. Seam fasteners
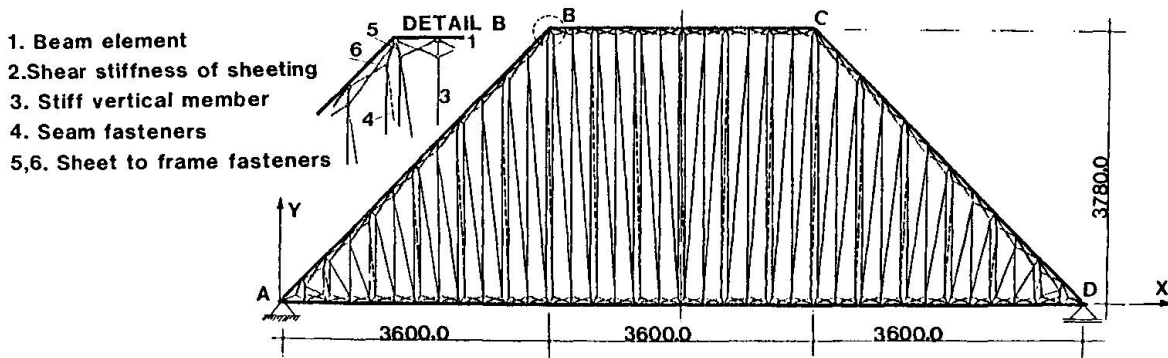5,6. Sheet to frame fasteners

Fig. 8   Full truss simulation of MACE panel

The flexibility c of a complete diaphragm can be calculated as the sum of the flexibilities of the various components including such factors as distortion of the sheeting profile and movement in the various fasteners [7]. If a very simple truss model is used, such as is shown in Fig. 9, the total flexibility of all of the components can be simulated by an arrangement of diagonals and relatively stiff members in the y direction.

It is easy to show that the necessary cross sectional area A_d of the diagonals is given by

$$A_d = \frac{L_d}{Ec \cos^2 \theta}$$

where L_d is the length of the diagonal and θ its inclination to the y axis. Before this simplified model can be incorporated into a three-dimensional analyses, its

Fig. 9   Simple truss simulation

accuracy requires detailed investigation. A comparison was therefore made between the performance of the three alternative models of the MACE panel with both pinned and rigid joints between the perimeter members for the following load cases:

- uniformly distributed load on eaves member AD
- point loads in y direction at B and C
- point load   in y direction at B only
- point load   in x direction at B

Full details of this investigation are given in [5].

Fig. 10   In-plane deflections (mm) from alternative analyses of the MACE panel - (a) bottom flange, (b) top flange.

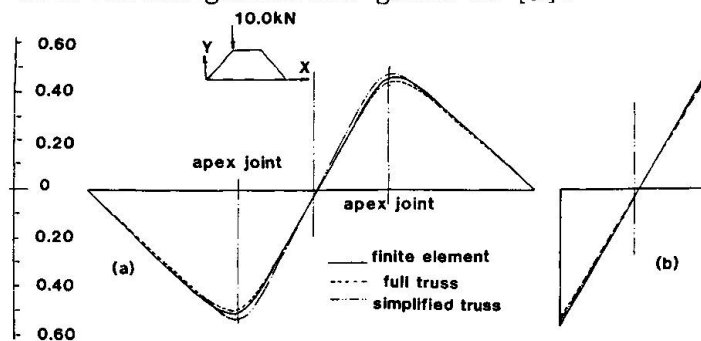A typical comparison of deflections is shown in Fig. 10 and it is clear that the simple truss model gives an accurate estimate of the deflected shape. However, it is known from earlier work [2] that large forces in the sheet to edge member fasteners occur at the junction between the hip and the upper horizontal members. A typical distribution of fastener force in the critical region is shown in Fig. 11. This was obtained from an analysis of the MACE panel with pinned joints and with a distributed load of $2.6kN/m^2$ on the bottom flange. If the four fasteners at and adjacent to the joint have forces F, F ,F^1,F_1^1, then the internal vertical force resultant

$$V = F_D \cos \theta - F_V$$

can be equated to the approximate resultant to the fastener forces

$$V_p = F + F_1 + (F^1 + F_1^1) \frac{a-p}{a}$$

It then follows that the maximum fastener force may be obtained from the internal forces in the simple truss model as $F_{max} = VF/_{V_p}$

F = 2.42kN
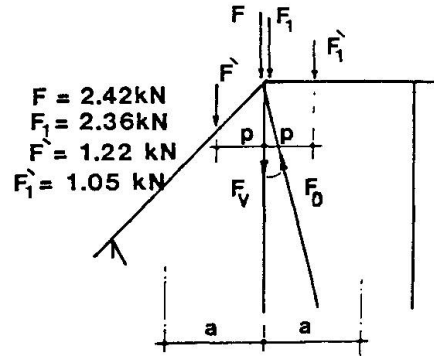F_1 = 2.36kN
F = 1.22 kN
F_1 = 1.05 kN

Fig. 11 Fastener forces in critical region of MACE panel

Detailed investigation of the MACE panel [5] shows that this approach gives consistent results for the four loading cases considered and that, for practical purposes, it is sufficient and safe to neglect F^1 and F_1^1 and take $F_1 = F$.

However, the MACE panel had sheet to edge member fasteners in alternate corrugation troughs at 300mm centres whereas, in the Pyradome, these fasteners were in every trough at 150mm centres. While the above approach may be expected to give safe answers for this case, it is likely to prove very conservative.

2.2 Models for the complete structures
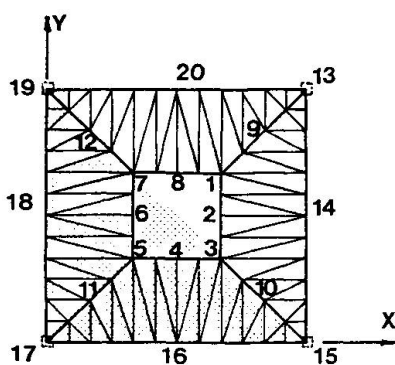
Fig. 12 Plan of model for analysis of MACE unit.
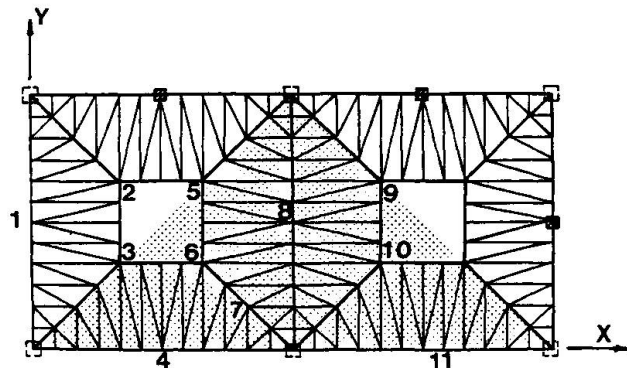
Fig. 13 Plan of model for analysis of Pyradome.

It follows that a suitable model for the analysis of the full MACE unit is shown in plan in Fig. 12. An assembly of two such units was used to analyse the Pyradome as shown in Fig. 13. In each case, the shaded area is the area loaded in the assymmetrical load condition and the numbers refer to those of the load measurement positions for which values are quoted later. It should be noted that the support conditions for the Pyradome are such that the two bays are not identical.

3.   COMPARISON OF TEST RESULTS WITH THEORY

In each case, the theoretical analyses were performed twice, once with hinged joints between the framing members and once with rigid connections.

3.1   Comparison between theory and experiment for the MACE unit

Under a uniformly distributed load of 0.8kN/m^2 over the entire roof, the comparison between the experimental and theoretical deflections is given in Table 1. Here, the stressed skin action is of a secondary nature and the generally higher experimental deflections may be attributed to movement in the bolted joints between the framing members. However, the vertical movement at point 9 is restrained by stressed skin action and it is interesting to note that the experimental result falls between the two alternative analyses. The calculated maximum fastener forces are 0.3kN (hinged joints) and 0.09kN (rigid joints) and are not critical.

Location	Experiment	Theory	
		Hinged	Fixed
1z	12.6	9.26	9.06
2z	13.9	12.6	10.8
9z	7.5	10.5	5.75
13x	0.92	1.05	1.02
14x	4.17	3.5	3.35

Location	Experiment	Theory	
		Hinged	Fixed
1	-21.0	-85.1	-21.0
2,8	-12.5,-13.2	-39.0	-10.7
3,7	12.4, 8.77	6.86	6.79
4,6	34.5, 34.9	57.6	26.9
5	55.2	98.8	34.5
9	-11.2	-42.7	-15.2
10,12	7.15, 6.51	7.81	4.31
11	39.9	58.3	23.8

Table 1   Deflections (mm) for the uniformly loaded MACE unit (x horizontal, z vertical)

Table 2   Vertical deflections (mm) for the assymmetrically loaded MACE unit

Under assymmetric loading of 1.21kN/m^2 over half of the area of the roof, the comparison between the experimental and theoretical deflections is given in Table 2. The experimental deflections are mostly between the two theoretical limits. The calculated maximum fastener forces are 4.23kN (hinged joints) and 0.70kN (rigid joints). Whereas the latter are quite acceptable, the values obtained with hinged corner joints exceed the capacity of the connection and the difference between the two is too large for a reasonable estimate to be made. The diaphragm behaviour is clearly very sensitive to the assumptions made regarding joint rigidity.

3.2   Comparison between theory and experiment for the Pyradome

In Table 3, a comparison is given between the experimental and theoretical deflections for two load cases, namely:
        Case A -   uniform load of 0.80kN/m^2
        Case B -   uniform load of 0.80kN/m^2 over the shaded area in Fig. 13, the remainder of the roof carrying 0.40 kN/m^2.

Here, with sheet to edge member fasteners in every corrugation trough, the theoretical results for hinged and fixed joints are generally closer together. Bearing in mind the factors contributing to the deflection pattern, the comparison between the theoretical and test results is, with the exception of location 8, considered to be satisfactory.

Approximate investigation of the fastener forces using the procedure indicated in section 2.1 showed these to be generally within acceptable limits. However, individual fastener forces of the order of the ultimate load were predicted in the diaphragms adjacent to the central valley for both loading cases and it is likely that non-linear behaviour of these fasteners contributed to the large deflections at location 8. This problem has been resolved by subsequent development which includes the provision of eaves and valley

members which can span 10 metres without the assistance of diaphragm action.

Location	CASE 'A'			CASE 'B'		
	Experimental results	Theoretical results		Experimental results	Theoretical results	
		Hinged	Fixed		Hinged	Fixed
1	12.0	16.0	8.31	5.50	8.0	4.06
2	7.0	7.28	7.19	3.50	-0.94	2.0
3	7.0	7.96	7.47	4.50	6.24	5.44
4	12.0	16.0	5.17	12.0	16.0	5.19
5	9.0	9.22	7.77	5.0	9.18	6.55
6	9.0	8.16	7.52	9.0	10.9	9.45
7	8.50	9.63	5.35	8.50	11.0	6.81
8	19.5	11.7	9.86	18.0	10.6	8.8
9	9.50	10.0	8.24	5.50	8.70	6.76
10	9.0	8.0	8.02	8.0	11.8	9.85
11	12.0	16.0	5.40	16.5	16.0	5.31

Table 3.   Vertical deflections (mm) for the Pyradome

CONCLUSIONS

- The simple truss simulation provides a practical and sufficiently accurate method of analysis for three-dimensional stressed skin assemblies.

- The analysis is sensitive to the assumptions made regarding the stiffness of the joints between the framing members, particularly for diaphragms where the sheeting is fastened to the framing members through alternate troughs and this condition should be avoided unless the forces arising as a result of diaphragm action are small.

- Fastener forces can be critical where the framing members meet. A suitable method of estimating these forces has been given.

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