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The Design and Testing of a Steel Building Taking Account of the Sheeting

Projet et essai d'une construction en acier, compte tenu du revêtement

Entwurf und Versuch an einer Stahlbaukonstruktion unter Berücksichtigung der Verkleidung

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1. Introduction

The conventional method of designing steel framed buildings is to assume that all the load is carried on the bare frames acting individually. It neglects the integral action of the building as a whole. In fact, the ability of roof and floor panels to resist shear in their own planes provides a stressed skin construction which causes the sheeted frames to behave very differently from that assumed by conventional theory. Stressed skin principles have been used in the design of aircraft, ships and cars, but until recently they have not been used in buildings.

In order to establish a theory for the stressed skin design of buildings, it is necessary to first study the individual behaviour of the interacting components. The elastic and plastic behaviour of bare steel frames has been the subject of exhaustive research and is now well established. The other interacting component, the sheeted panel, has been the subject of theoretical and experimental investigation during recent years. In the United States, the work has resulted in a design method for specific types of construction (1), and in Britain a general theory has been developed (2) which gives very satisfactory results when applied to panels using different types of sheets and fasteners. This work is to be published shortly as a design manual (3).

2. General Principles of Design

Most buildings have two or more frames which are braced or sheeted in their own planes eg at the gable ends. In such buildings, the differential flexibility between the gable frames and intermediate frames will result in interaction between the frames and roof panels. The roof sheeting or decking will act like the web of a deep plate girder spanning from gable to gable and will transfer part of the spread or sway forces from the intermediate frames to the gable ends. Fig 1 illustrates the effect in a rectangular building under side load.

The extent of the interaction between the frames and panels depends on the ratio of panel shear flexibility to bare frame flexibility. For a rectangular portal frame, the bare frame flexibility is defined as the eaves displacement k per unit side load (Fig 2a)

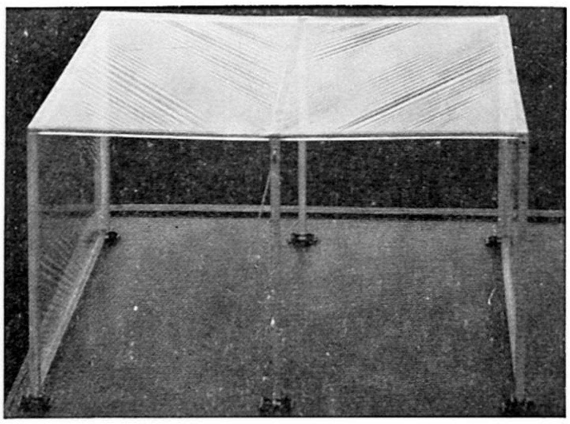

Fig 1. Membrane action in a rectangular building.

and for a pitched roof portal frame, the bare frame flexibility may be subdivided into that due to sway k_{sw} (Fig 2b) and that due to spread k_{sp} (Fig 2c). The panel shear flexibility is defined as the shear displacement c of a panel of sheeting or decking per unit shear load (Fig 3). The effect of the interaction is to reduce the moments and deflections in the frames.

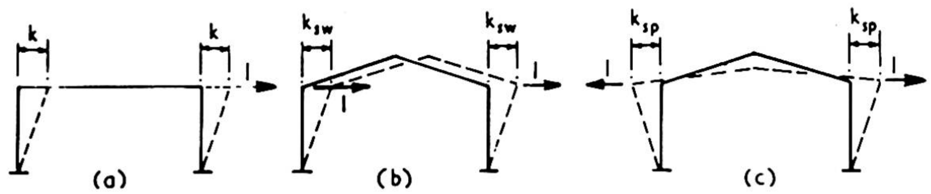

FIG. 2. FRAME FLEXIBILITY

Previously tests have been carried out on full scale sheeted buildings to study the effect of sheeting on bare frames which have been designed in accordance with conventional methods (4). The purpose of the tests described in the present paper is different in that the sheeted building was designed in advance according to stressed skin theory. The philosophy on which the theory is based ensures the safety of the building at all stages of construction and use, and is given as follows:-

2.01 The steel framework must be strong enough by itself to withstand all the design loads. Under this condition, the maximum stress in the frames may be allowed to approach the guaranteed yield stress.

2.02 The effect of the sheeting will be to provide the factor of safety required by the various national standards.

This philosophy simplifies design and ensures the safety of the building at all times, even if sheeting is removed from the completed building.

3. Description of Test Building

The arrangement of the test building is shown in Fig 4. The six main frames were pinned base, pitched roof portals of 12m span and 6m spacing, so that the size of the building in plan was 12m x 30m. The columns were of 178mm x 102mm x 21.5kg/m

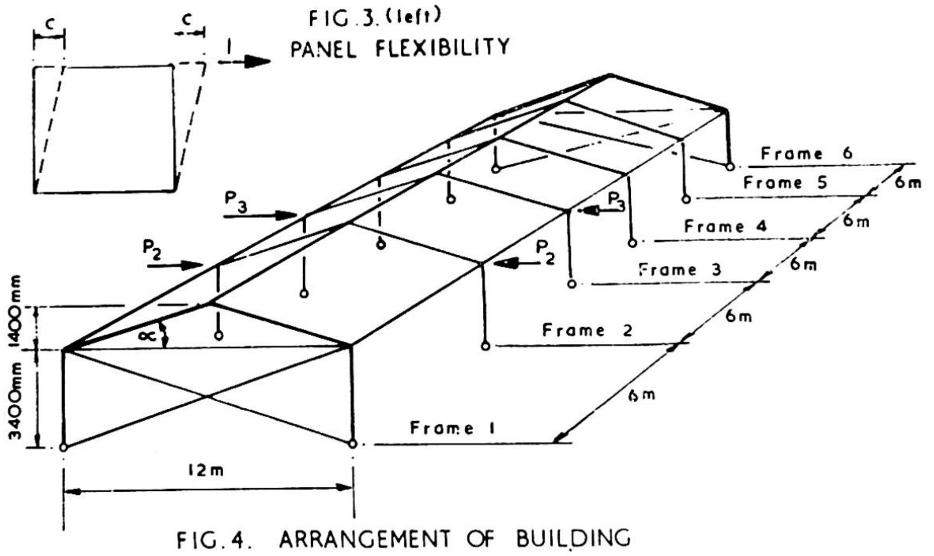

FIG.4. ARRANGEMENT OF BUILDING

joist section and the rafters were of 152mm x 89mm x 17.1kg/m joist. The purlins were spaced at about 1.5m centres and were of 140mm x 45mm x 2mm cold rolled Z section. The connections with the rafter were made through the flanges, without the use of angle cleats. Shear connectors made from short lengths of Z purlin (Fig 11) were used to transmit shear from the sheeting to the rafters and these also prevented twist in the purlin/rafter connection.

The roof was sheeted with 0.46mm thick plastic coated steel sheeting (Everclad L5 profile) interspersed with translucent sheeting of the same profile so that the area of the roof lights was 12.5 per cent of the roof area. The sheeting was fixed to the purlins with 6.1mm o.d. self tapping screws in every corrugation (15.2mm pitch) and the seams of adjacent sheets were fastened with 4.8mm aluminium pop rivets at 250mm centres. The sheeting was fastened to each shear connector with four 6.1mm o.d. self tapping screws.

The end gables of the building were braced, instead of sheeted, to allow access to the building during testing. In addition, the end bays of the building and the end roof panels were braced, in accordance with normal practice, to square up the building and provide stability during erection.

For comparison purposes, a separate steel frame, similar in all respects to an intermediate frame in the sheeted building, was erected and tested as a bare frame (Fig 7).

4. Design Criteria

The test building was designed for the superimposed loading specified for agricultural buildings, 480N/m^2, and the eaves deflection under a simulated wind load was checked. The design criteria, derived from the philosophy propounded in sections 2.01 and 2.02, were as follows:-

4.01 The stress in the bare frames was not to exceed the guaranteed yield stress of 247N/mm^2 assuming that all the design load was borne by the frames alone. The columns and rafters were also to satisfy the elastic stability criteria given in BCSA Publication No 23 using a load factor of unity.

4.02 In the sheeted building the stress in the frames was not to exceed the permissible working stress of $160N/mm^2$ and the load factor for member stability was to be not less than 1.75. For the sheeting and fasteners, the load factor was to be not less than 2.25.

4.1 Design of Bare Frame (criterion 4.01)

The design loads and elastic bending moment diagram of a bare frame are shown in Fig 5. The maximum moments in the column and rafter occur at the ends of the haunch and give rise to bending stresses of $196N/mm^2$ and $192N/mm^2$ respectively. Although these stresses are well below the guaranteed yield stress of $247N/mm^2$, and although the load factor against member instability is 1.54, the sections chosen are the smallest allowable to satisfy the criteria given in paragraph 4.01. This is because the choice of sections at the small end of the range is very limited.

4.2 Design of Sheeted Building (criterion 4.02)

In order to be able to check the design of the sheeted building it is first necessary to calculate the flexibilities of its components. The calculated sway flexibility of a bare frame is $k_{sw} = 15.2mm/kN$ and the calculated spread flexibility of a bare frame is $k_{sp} = 0.43mm/kN$. The calculated eaves sway of the bare frame under horizontal wind loads at the eaves of 2.5kN is $\Delta = 37.8mm$, and the calculated eaves spread of the bare frame under the design loads of Fig 5 is $\Delta = 17.8mm$ per eave.

Before the shear flexibility of a panel of sheeting can be determined, the parameters of the sheeting and fasteners have to be defined and their values listed. Using these values, the shear flexibility of a typical interior panel (eg panel 2-3) is calculated in accordance with the theory given in reference (3). From this, $c_{2-3} = 0.30mm/kN$. In the end panels (eg panel 1-2), the shear flexibility is modified by the erection bracing to $c_{1-2} = 0.18mm/kN$.

Let the design of the building under superimposed loads be considered first. From Fig 4 if P_2 and P_3 are the horizontal eaves forces exerted on frames 2 and 3 by the sheeting and if Δ_2 and Δ_3 are the eaves deflections of the sheeted frames 2 and 3,

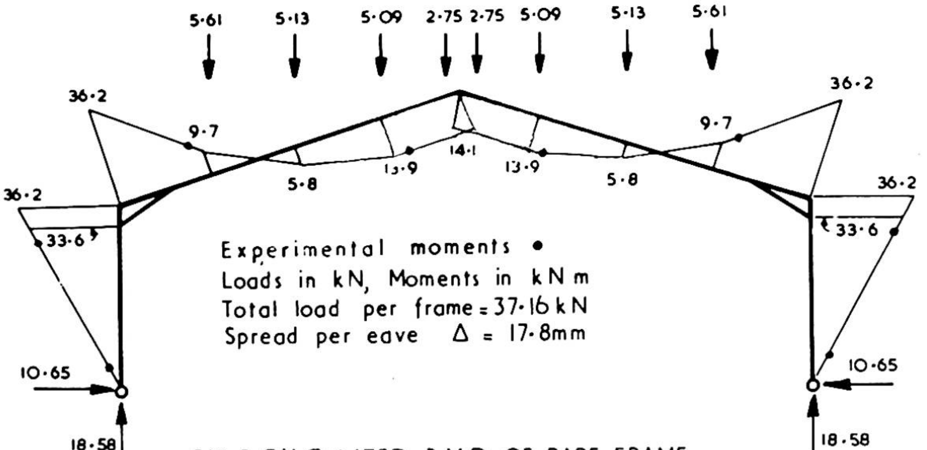

FIG 5. CALCULATED B.M.D OF BARE FRAME

then: -

For the frames, $\Delta_2 = \Delta - k_{sp} P_2$ ...... (1)

$\Delta_3 = \Delta - k_{sp} P_3$ ...... (2)

For the sheeting, $\Delta_2 \cos\alpha = c_{1-2}\left(\dfrac{P_2 + P_3}{\cos\alpha}\right)$ ...... (3)

$\Delta_3 \cos\alpha = c_{1-2}\left(\dfrac{P_2 + P_3}{\cos\alpha}\right) + c_{2-3}\left(\dfrac{P_3}{\cos\alpha}\right)$ ...... (4)

From equations (1) to (4), $P_2 = 24.3$kN, $P_3 = 14.0$kN, $\Delta_2 = 7.3$mm, $\Delta_3 = 11.8$mm

Under the action of these eaves forces, the bending moments in frames 2 and 3 of the sheeted building may be calculated to give the results shown in Fig 6. The maximum bending stress in frame 3 is 151N/mm^2 which is less than the design specification of 160N/mm^2 given in paragraph 4.02. In addition, the minimum load factor against member instability is 1.98 which is satisfactory.

Considering the sheeting and fasteners, it may be shown (3) that shear failure of an interior panel will occur at 44.1kN and will be due to tearing at the sheet/connector fasteners. This shear strength results in a load factor of 3.07 which is amply greater

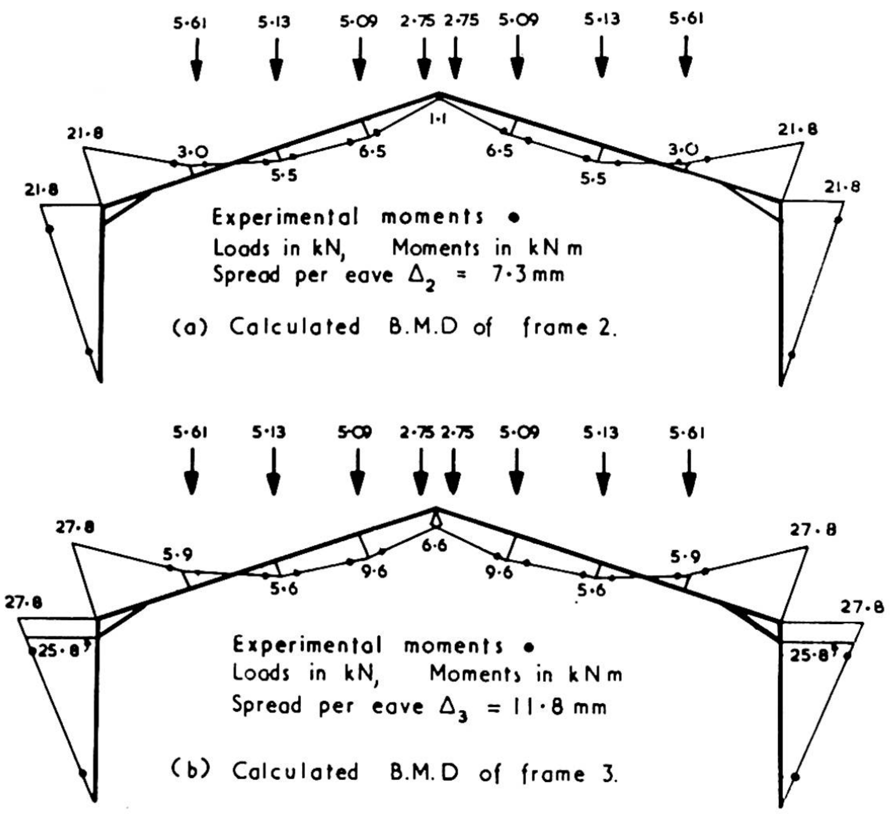

FIG. 6 CALCULATED B.M.D OF SHEETED BUILDING

than the value of 2.25 given in paragraph 4.02.

The behaviour of the sheeted building under wind load may be calculated using equations similar to (1) to (4) with k_{sp} replaced by k_{sw} and Δ replaced by the value for sway. These revised equations lead to a calculated eaves deflection at frame 3 of 1.7mm. If frame 3 were loaded by itself, the calculated eaves deflection would be 0.7mm.

5. Calculated Failure Loads of Frame

From control tests, the actual yield stress of the steel used in the frames was found to be 313N/mm^2. Using this value the failure loads of the bare frame and sheeted frame were calculated as given in Tables 1 and 2. In both the bare frame and sheeted frame failure should occur by collapse of the columns if the columns are laterally unsupported, or by simple plastic collapse of the frame if the columns are laterally supported.

6. Test Arrangements

6.1 Bare Frame

The superimposed load was applied at the purlin points of the bare frame through a steel linkage (Fig 7) using a 100kN jack. The side load was applied just below the eaves by means of a wire rope passing over a pulley and loaded with dead weights. Strains were measured by resistance strain gauges and deflections at the eaves and apex were measured with linear potentiometer transducers.

6.2 Sheeted Building

In the sheeted building, the superimposed load was applied through timber grillages (Fig 7) operated by 100kN hydraulic jacks fixed to the strong floor of the laboratory (Fig 8). The side load was applied as for the bare frame. The methods of measuring and recording strains were the same as for the bare frame.

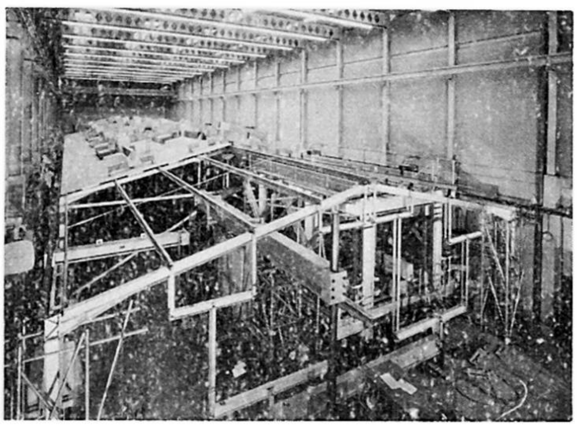

Fig 7. Loading arrangements for bare frame.

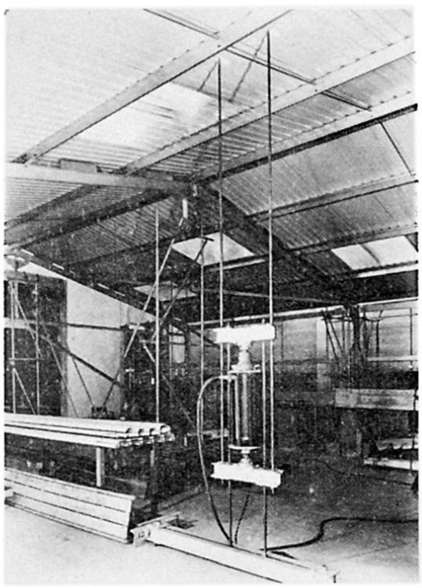

Fig 8. Jacking point.

7. Test Programme

Two series of tests (test series I and II) were carried out on the bare frame and sheeted building as follows: -

Test Series I - with the columns laterally unsupported

Test Series II - with the columns laterally supported at about two thirds of their height by rails.

In each series the following tests were carried out: -

7.01 Bare frame. Side loads at the eaves of 2.5kN.

7.02 Bare frame. Super loads at the purlin points increased to collapse.

7.03 Sheeted building. Side eaves loads of 2.5kN at each frame separately and then at all frames together.

7.04 Sheeted building. Super loads at each frame separately and then eventually increased to collapse at all frames together.

7.05 Sheeted building. Design super loads at all frames together. Loading and unloading for twenty cycles.

7.06 Sheeted building. Design super loads at all frames together. Sustained load for two days.

8. Test Results

8.1 Bare Frame

The results of the tests on the bare frame are summarized in Table 1. As expected there is close agreement between the calculated and observed deflections. There was also excellent agreement between the calculated bending moments and those derived from the strain gauge readings as shown in Fig 5.

	Calculated value	Observed value	Notes
1. Eaves deflection under side loads of 2.5kN	37.8mm	34.0mm	Test Series I
		31.7mm	Test Series II
2. Eaves deflection under design superimposed load	17.8mm	20.1mm	Test Series I
		18.2mm	Test Series II
3. Superimposed load to produce failure in columns	57kN	67kN	Test Series I
4. Simple plastic collapse load of frame under superimposed load	75kN	95kN	Test Series II

Table 1. Calculated and Observed Behaviour of Bare Frame

	Calculated value	Observed value	Notes
1. Maximum eaves deflection under side loads of 2.5kN (frame 3 only loaded)	0.7mm	0.9mm 1.1mm	Test Series I Test Series II
2. Maximum eaves deflection under side loads of 2.5kN (all frames loaded)	1.7mm	3.3mm 3.1mm	Test Series I Test Series II
3. Maximum eaves deflection under design superimposed load	11.8mm	16.3mm 14.3mm	Test Series I Test Series II
4. Superimposed load to produce failure in columns	73kN	82kN	Test Series I
5. Simple plastic collapse load of frame under superimposed load	110kN	122kN	Test Series II

Table 2. Calculated and Observed Behaviour of Sheeted Frame.

In Test Series I, with unsupported columns, failure occurred by column instability and in Test Series II, with supported columns, failure occurred by plastic collapse. In each case failure occurred in the predicted mode.

8.2 Sheeted Building

The results of the tests on the sheeted frames are summarized in Table 2. If allowance is made for the fact that the end gables were not completely rigid, as assumed in the calculations, then there is satisfactory agreement between the calculated and

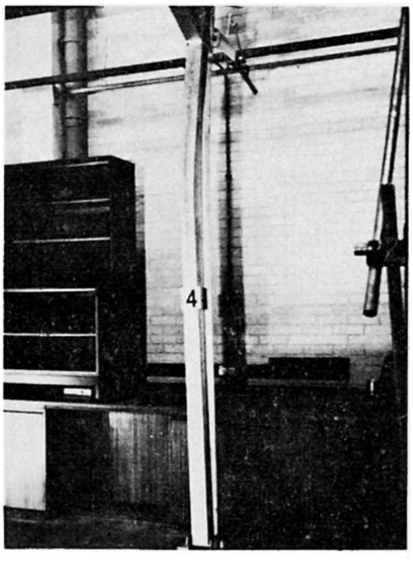

Fig 9. Instability in unsupported column in sheeted building

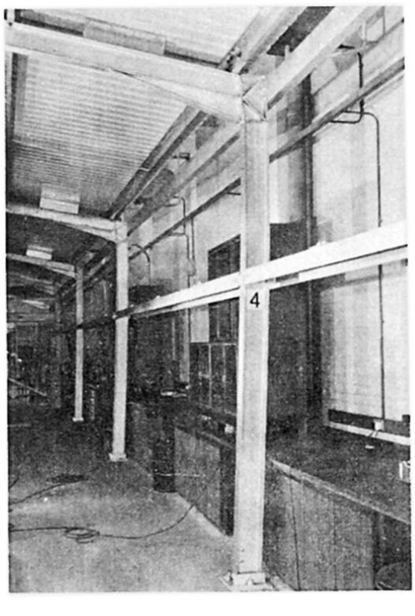

Fig 10. Plastic collapse in supported column in sheeted building

Fig 11. Plastic collapse of sheeted building.

observed deflections. There was also excellent agreement between the calculated bending moments and the measured moments as shown in Fig 6.

In Test Series I, with unsupported columns, collapse of the building started in frames 3 and 4 and extended to frames 2 and 5. The cause was column instability (Fig 9) which occurred immediately after the formation of a plastic hinge in the column below the haunch. After collapse, the sheeting and fasteners were inspected and found to be in perfect condition, but nevertheless the seam fasteners and shear/connector fasteners were replaced before Test Series II was carried out. In addition, all the buckled columns were replaced.

In Test Series II, with supported columns, simple plastic collapse of the frames commenced at the middle of the building (at frames 3 and 4, Fig 10) and extended towards the ends (to frames 2 and 5). The effect was therefore one of three dimensional collapse (Fig 11). After collapse, it was found that sheet failure had occurred by tearing at the sheet/connector fasteners at several frames. In both Test Series I and II, failure occurred in the mode predicted.

With regard to the tests listed in 7.05 and 7.06, ie loading and unloading cycles, and sustained load, the sheeted building behaved in an entirely satisfactory manner. At the design loads there was no evidence of shakedown in the building, nor of creep. It is therefore evident that these effects would not influence the design of the test building.

9. Conclusions

1. Compared with a conventional design, the stressed skin design of the building described saves 25 per cent of the weight of structural steel in the main frames. In view of the thinness of the sheeting, and low pitch of the roof, this is a considerable achievement.

2. From the tests under side load, it is apparent that the sheeting almost entirely eliminates sidesway in the frames. This fact could be a very important design consideration.

3. Although the sheeting was fastened to the purlins in every corrugation, the sheeting would have provided virtually as much restraint if it had been fastened to the edge purlins at every corrugation and to the intermediate purlins at alternate corrugations. This degree of fixity is quite common in roof sheeting as high suctions have to be withstood at the eaves

and ridge. Hence stressed skin design may involve little, if any, additional fasteners.

4. The test building illustrates one application of stressed skin design. Perhaps an even wider application is in roof and floor decks in flat roofed buildings. In these cases, the decking will be extremely effective in resisting horizontal forces.

5. One of the most important features of the work is that the design method reflects the actual behaviour of the complete building, not just the bare frame. Whether interaction effects are taken into account in design or not, they are bound to occur in practice; consequently it is only logical that they should be considered in the design method.

10. Acknowledgements

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SUMMARY

The interaction between the sheeting and frameworks of buildings may be taken into account in design to give a stressed skin design. The philosophy and principles of such design are discussed and applied to a 12m span by 30m long steel sheeted, steel framed pitched roof building. The building in question is tested under horizontal and superimposed loads and its performance is found to be closely in agreement with that predicted by stressed skin theory. The design saves 25 per cent of the weight of conventional frames and reflects the actual behaviour of the building which is very different from that of the bare frames.