Jun 1st, 12:00 AM

Tests on a Continuous Purlin Roofing System

J. Rhodes

William King

Keith James

James M. Harvey

TESTS ON A CONTINUOUS PURLIN ROOFING SYSTEM

by

James Rhodes *
William King **
James M Harvey +
Keith James ++

SUMMARY

Details are presented of tests carried out to simulate the effects of live loading and wind loading on a continuous purlin roofing system. The results of the tests are presented and discussed in detail.

INTRODUCTION

In the introduction of a roof system employing continuous Z purlins clad with sheeting panels, several possible design problems were anticipated by the engineers of Butler Buildings (UK) Ltd. After discussion of these problems at the University of Strathclyde, it was decided that a realistic assessment of the system could best be achieved by test.

The system under question consists of Z purlins of 7.2 metre span, continuous over the supports, and joined to the sheeting majors by screws, via a clip arrangement, every 600 mm. Continuity is achieved by overlapping the purlins, at the supports, using a total lap length of 600 mm for intermediate frames. No cleats are used at the supports.

* Senior Lecturer Dept of Mechanics of Materials University of Strathclyde Glasgow Scotland
** Lecturer Dept of Mechanics of Materials University of Strathclyde Glasgow
+ Professor of Mechanics of Materials University of Strathclyde Glasgow
++ Chief Engineer-Director Butler Buildings (UK) Ltd

Sag bars are used at centre span. Details of purlins, sheeting and connections are shown in Fig 1.

The main points which were to be investigated can be briefly summarised as follows:

i) Would the absence of cleats at the supports give rise to problems of purlin roll or web crippling?

ii) Were the lap lengths adequate to avoid premature failure at the sudden change in section at the lap end?

iii) Were the sheet/purlin connections adequate to restrain the purlins laterally?

iv) Would the system adequately withstand wind uplift loading?

Two separate tests were undertaken to answer these questions, one designed to simulate live loading on a roof and the other to simulate wind loadings.

LIVE LOAD TEST

The system tested in this case consisted of three purlins continuous over two spans and having cantilever overlaps at the end supports, to enable fixing moments to be applied at these supports. A diagrammatic arrangement of the set up is shown in Fig 2. The roof was some 2 m high, purlin spans were 7.2 m and centres 1.5 m. The end cantilevers were originally designed to be two metres long, but restriction of space made it necessary to cut one overhang to one metre.

The object of this set up was to permit the testing of the central purlin only, under fully continuous conditions at the supports, with known loading applied. The relevant live load design condition is 1000 N/m^2.

The loading was applied using sand bags of weight 60 lb (267N) and hydraulic jacks. The sand bags were applied directly to the sheeting in predetermined positions to produce an approximate U.D.L. over the two spans. Hydraulic loading was supplied via 8 one ton (10 kN) jacks of 10 in (25.4 cm) stroke, each of which was fixed to the test floor and connected, using a spreader beam/chain arrangement, to four angle sections positioned on top of the sheeting mid-way between the central purlin and the edge purlins. Each angle section was supported on two sand bags through which the load was transmitted to the sheeting. A sketch of a typical jack loading system is shown in Fig 3 and the complete loading arrangement is as indicated in Fig 2.

Thus the load applied by each jack was distributed equally to eight areas of roof sheeting, so that with eight jacks fed from a common supply the roof was subjected to 64 patch loads, each of the same magnitude. Preliminary calculations showed that this loading should produce results very close to those obtained by a perfect UDL.

Fixing moments at the end supports were provided by laying sand bags on the sheeting directly over the ends of the central purling overhangs, the weight of bags being calculated to produce full fixing at each load increment.

During the test measurements were taken of deflections and strains at various points on the centre purlin. Deflections were measured by 2 in travel mechanical dial gauges and strains were measured using electrical resistance foil strain gauges.

The loading procedure was as follows. Initial readings were taken on all gauges with only the weight of the hydraulic loading tackle acting on the roof. Thereafter 32 sand bags were laid on the roof, in staggered positions so that an approximate UDL was obtained, and additional bags laid at the end of the cantilever overhangs to supply fixing moments at the end supports. Readings of all gauges were taken. Two further loading increments (32 sand bags each plus cantilever loads) were applied with readings taken after each increment. At this point the sand bag loading was complete and further loads were applied hydraulically. These were applied in increments of 100 lbf/in^2 pressure to each jack, giving 8 jack loads of 218 lb (970 N) at each increment. Additional cantilever loads were applied at each increment to retain full fixing of the end supports, and strain and deflection readings were taken for each load increment. Loading was continued until the hydraulic pressure reached 600 lbf/in^2. At this point the total load was of the order of twice the design load and was considered sufficient to prove the roof system's efficiency. The load was then removed and readings taken of the residual strains and deformations.

Some aspects of the tests are shown in Figs 4 and 5. Fig 4 shows a view under part of the roof in which some of the spreader beams can be seen prior to fitting the jacks. Light slotted angle frameworks carrying the deflection gauges can also be seen in the figure. Fig 5 gives a view of the roof from above during the test and shows the layout of the loading on the roof.

WIND UPLIFT TEST

The system in this case consisted of three purlins on a single span, with two metre cantilever overlaps at each support to provide for the application of fixing moments. The span was again 7.2 m and purlin centres 1.5 m. The test set up is shown in Fig 6.

As in the previous test the object was to test the central purlin, only under fully continuous conditions. Loading was again applied using sand bags and hydraulic jacks.

In order that a uniformly distributed wind uplift load be simulated the roof structure was inverted as shown in Fig 6. The purlins were attached to the supporting frame, the sheeting was then attached, and the complete roof was then turned upside down before being fixed to the supporting joists. Thus the application of vertically downwards loads to the sheeting had the tendency to separate the sheeting from the purlins as in wind suction loading.

In this case 5 hydraulic jacks were used within the span and two others were connected to the central purlin at the ends of the cantilever overlaps to help in achieving the fixing moments. The loading arrangement is also shown in Fig 6 (plan view). Calculations were made to ensure that the distribution of loading resulted in one which closely approximated a UDL.

As in the previous test, mechanical deflection gauges and electrical resistance strain gauges were positioned at various points on the central purlin.

The loading procedure was similar to that of the previous test. Initial readings were taken with only the weight of the loading tackle acting. A first increment of 26 sand bags was applied within the span and 6 sand bags were placed on the cantilevers to provide fixing moments, and readings were taken. A second increment of 24 sand bags plus fixing moments was applied and readings again taken. Subsequent loadings were applied hydraulically in increments of 100 lbf/in^2. Since the jacks attached to the cantilever ends could not apply sufficient load to achieve full fixation at the supports, the deficit was made up with sand bags.

The hydraulic loading was increased incrementally until failure occurred at a pressure of 450 lbf/in^2. The failure occurred on the cantilever outwith the span, while that part of the system within the span remained capable of carrying load.

Fig 7 gives a view under the roof showing the set-up of the loading jacks and Fig 8 shows the top of the roof during loading.

RESULTS OF LIVE LOAD TEST

The design condition for the roof system tested is 1000 N/m^2 live load with purlins spaced 1.5 m apart, and having 7.2 m span. Using this information the total design load, P_D, to be carried over two spans by a single purlin and associated sheeting is 4854 lbf (21.6 kN).

The main object of the tests on the three purlin system examined was to compare the load taken by the central purlin with the specified design load. It is therefore necessary to determine the proportion of the total applied load which is taken by the central purlin. Determination of the central purlin loading is rather complex and depends very much on the relative stiffnesses of purlin and sheeting. For example, if the flexural rigidity of the sheeting is assumed to be negligible and the purlins assumed completely rigid, then simple statics show that the centre purlin takes half of the total load on the roof. This is also the case in normal design conditions when a number of purlins are under load. If the flexural rigidity of the sheeting is taken into account for the three purlin system and the purlin flexibility is neglected, the central purlin takes $\frac{5}{8}$ of the total load. However, if the purlin deflections are very much greater than those of the sheeting then the load on the central purlin reduces to $\frac{1}{3}$ of the total load. Thus to obtain a realistic assessment of the central purlin loading a deflection analysis must be performed, which takes into account the relative stiffnesses of sheeting and purlins. Such an analysis was carried out and is outlined in the Appendix. On this basis a value of 0.558 times the total load was considered to constitute a close approximation to the load applied to the centre purlin/sheeting combination.

Fig 9 shows the variation in horizontal and vertical deflections with load for various points on the centre purlin. The vertical deflections are seen to vary in an approximately linear fashion with load, indicating that the behaviour was completely elastic. Also shown is a line described as the theoretical centre deflection. This line was drawn on the basis that the centre purlin behaved as a fixed-fixed beam carrying 0.558 of the total applied load, as discussed previously. The close agreement between this line and the experimentally obtained centre deflection (gauge 5) substantiates to some extent the actual fixity conditions achieved at the supports and the assumptions made in estimating the central purlin loading.

The horizontal deflections were very small and somewhat random in their variation, apart from those measured at the quarter point. It would appear from this that no danger of lateral buckling was present, even using sag bars only at mid-span. The 'more or less' linear growth in horizontal deflections at the quarter point is to be expected due to the tilt of the natural neutral axis of the section. As can be seen, this tendency to horizontal deflections was adequately resisted at the centre by the sag bars.

Figs 10 and 11 show typical variations in strains at different cross-sections of the centre purlin. These variations were obtained on the basis of 4 electrical resistance gauges situated at the junctions of web and flanges and flanges and lips, as indicated in the figure, to measure strain in the longitudinal direction. These results indicate that the purlin was constrained to bend about an axis between the neutral axis and the x-x axis. Thus help was being obtained from the sheeting in withstanding the applied loading.

By converting the strains to stresses, the results were used to estimate the applied moments at the strain gauged sections. A typical comparison of the moments so evaluated with the bending moment diagram to be expected theoretically from a fully fixed centre purlin carrying 0.558 of the total loading is shown in Fig 12.

The experimentally derived moments are less than those suggested theoretically. This indicates that the sheeting withstands a substantial part of the loading; some 20% being a reasonable estimate.

The experimental variation in moments at different sections of the beam also indicates the degree of fixing obtained at the supports.

On removal of the loading from just over twice the design condition recovery on the various deflection gauges varied from 67% to greater than 100%. Recovery on the strain gauges was of the same order. On the basis of the strain results it would appear that any lack of recovery was due to frictional effects at bolts, etc. as the highest strain value recorded at 2.05 times the design load was equivalent to a stress of 296 N/mm^2; i.e. 0.74 times the yield stress. It was therefore unlikely that any large degree of yield was present at any point on the purlin.

RESULTS OF WIND LOAD TEST

In this test the roof was loaded until failure, which just exceeded twice the design load, based on the factor of 0.558 previously discussed. Deflection readings taken at various points on the central purlin are shown in Fig 13. These show that lateral movements were small at loads up to 1.88 times the design load, the highest loads at which deflections were measured. It can be postulated that, although the sheeting did not restrain the purlins laterally in this case, as it did in the live load test, the application of load through the connections to the tension flange reduces greatly the chances of lateral buckling of the purlin in comparison to compression flange loading.

Again the central vertical deflection agreed closely with that obtained from simple beam theory.

Typical strain results are shown in Figs 14 and 15. In this test the strain patterns obtained were different at different sections of the purlin. On two sections, at the lap ends, the purlin appeared to be bending as an unrestrained beam about its natural neutral axis, as shown in Fig 14, whereas rather singular strain variations were observed for the sections at mid-span (Fig 15) and at the support. Recalling that connections were present at mid-span and at support points gives an indication of the reason for the singularities. At these points it would appear that the concentrated load applied through the connecting screw has the effect of twisting the purlin in the opposite direction to that in which it is naturally inclined to twist, possibly imposing distortion on the flange as a result, and causing high stresses at the lips near the connection points. This effect acts against lateral instability since a restraining torque is thus applied to the purlin.

The variation of moment calculated from the strain results is compared with simple beam theory, in Fig 16, again indicating somewhat lower experimental values. This figure also indicates that fixity at one end was marginally greater than at the other, and indeed greater than was required. When failure did occur it occurred outside the main span on the cantilever arm, thus giving further substantiation to this suggestion. Checking the final two sets of strain results and extrapolating these to the failure load shows that the maximum recorded strain at failure, based on linear extrapolation, was within 7% of yield. Since undoubtedly points of higher strain existed it is unlikely that the recorded failure load would have been much greater if the cantilever arm had not given way.

CONCLUSIONS

In general the results of the tests showed that the roofing system adequately resisted loads substantially greater than the design load under both live load and simulated wind uplift load conditions. It is of note that although sag bars were used only at mid-span the purlins showed very little tendency to buckle laterally during either live load application or simulated wind loading.

The questions posed in the introduction are now answered by the following conclusions obtained:-

1 The live load test indicated that under the conditions investigated the absence of cleats at the supports caused no problems.

2 The overlap lengths used did not initiate premature failure, although the highest strains were encountered at the lap ends.

3 The sheet/purlins connections behaved adequately in restraining the purlins.

4 The system under test withstood simulated wind uplift loading of approximately twice the design load.

APPENDIX

Determination of equivalent loading on central purlin/sheeting combination

The following analysis is based upon a number of approximations, since a completely rigorous investigation of this problem would be extremely lengthy.

The following assumptions are made:

1) The purlin deflections are calculated on the basis of uniformly distributed loading on fully fixed purlins.
2) The sheeting deflections for the first test are calculated on the basis of uniformly distributed loading on a beam with three elastic supports.
3) The sheeting is fixed rigidly to the purlins at the majors and loads are applied through the majors.

The deflections of purlins and sheeting majors are shown diagrammatically in Fig A1.

Consider the deflections of the purlins and sheeting major shown in section X-X. From a study of the sheeting deflections only, the load taken by the central purlin can be expressed as follows:

$$R = (\tfrac{5}{8}W - 48\tfrac{EI}{\ell^3}(\Delta_B - \Delta_A)) \quad (1)$$

where R is the load taken by the central purlin, EI is the flexural rigidity of the sheeting and ℓ is the total distance between the outer purlins.

Also, since all purlins are the same length the deflection of each purlin is proportional to its load, so that

$$\frac{\Delta_B}{\Delta_A} = \frac{R}{\frac{1}{2}(W-R)}, \text{ or } \frac{\Delta_A}{\Delta_B} = \frac{W-R}{2R} \quad (2)$$

Therefore the term $\Delta_B - \Delta_A$ in equation (1) can be written

$$\Delta_B - \Delta_A = \Delta_B(1 - \frac{\Delta_A}{\Delta_B}) = \Delta_B(1 - \frac{W-R}{2R}) = \Delta_B\frac{(3R-W)}{2R}$$

Equation (1) now becomes

$$R = (\frac{5}{8}W - \frac{48EI}{\ell^3} \; \Delta_B \frac{(3R - W)}{2R}) \tag{3}$$

Now it only remains to specify the deflection of the central purlin, Δ_B , in terms of R to obtain the load R applied through any particular sheeting major. At the supports Δ_B=0 and R = $\frac{5}{8}$W. At centre span the deflection is

$$\Delta_B = \frac{1}{384} \; (12R) \; \frac{L^3}{EI^*} \tag{4}$$

where 12R is the total load applied to the centre purlin via 12 majors, L is the purlin span and I^* the moment of inertia of the purlin. Substituting in (3) gives

$$R = (\frac{5}{8}W - \frac{3}{4}(\frac{L}{\ell})^3 \times \frac{I}{I^*}(3R - W)) \tag{5}$$

Rearranging gives

$$R = \frac{\frac{5}{8}W\left[1 + \frac{6}{5}\frac{I}{I^*}(\frac{L}{\ell})^3\right]}{1 + \frac{9}{4}\frac{I}{I^*}(\frac{L}{\ell})^3} \tag{6}$$

Substituting values of I = 10.835 cm^4, I^* = 343.81 cm^4, L = 7.2m and ℓ = 3m (values of I and I^* supplied by Butler Buildings) gives, for centre span

$$R = 0.481 \; W$$

Thus the proportion of load taken by the centre purlin at centre span is substantially less than that taken at the supports. To investigate the variation of central purlin load along the span the deflections of several points on the purlin were calculated and substituted into equation (3), giving the variation shown in Fig A2.

The effects of flexibility at the clips would be to add to the sheeting flexibility and increase the central purlin loads, although this is counteracted by the fact that restraints less than full fixity at the purlin supports would tend to reduce central purlin loads. In view of these effects it would appear reasonable to assume a central purlin uniformly distributed loading of magnitude obtained using the average of 0.625W and 0.481W, i.e. R = 0.558W.

In the wind uplift test loads were applied directly to the sheeting majors mid-way between two purlins so that the equivalent equation to (1) in this case is

$$R = \left(\frac{11}{16}W - 48\,\frac{EI}{\ell^3}\,(\Delta_B - \Delta_A)\right) \tag{7}$$

Using this in the subsequent analysis gives, instead of (6)

$$R = \frac{\frac{11}{16}W\left(1 + \frac{12}{11}\,\frac{I}{I^*}\,\left(\frac{L}{\ell}\right)^3\right)}{1 + \frac{9}{4}\,\frac{I}{I^*}\,\left(\frac{L}{\ell}\right)^3} \tag{8}$$

The average value of R obtained in this case is R = 0.597W. However, since in this case the flexibility of the clips would reduce this value, it is felt that the value used for the first test would more realistically apply here also.

Figure Captions

Fig 1 Detail of roofing systems

Fig 2 Test rig and loading arrangement (live load test)

Fig 3 Hydraulic jack/spreader beam arrangement

Fig 4 Underside view of test roof (live load test)

Fig 5 Top view of test roof (live load test)

Fig 6 Test rig and loading arrangement (wind uplift test)

Fig 7 View under test roof (wind uplift test)

Fig 8 Top view of test roof (wind uplift test)

Fig 9 Central purlin deflections (live load)

Fig 10 Strains recorded at mid-span of purlin (live load)

Fig 11 Strains recorded at lap end over central support (live load)

Fig 12 Bending moment distribution on purlin (live loads)

Fig 13 Central purlin deflections (wind load)

Fig 14 Strains recorded at lap end (wind load)

Fig 15 Strains recorded at mid-span of purlin (wind load)

Fig 16 Moment distribution along central purlin (wind load)

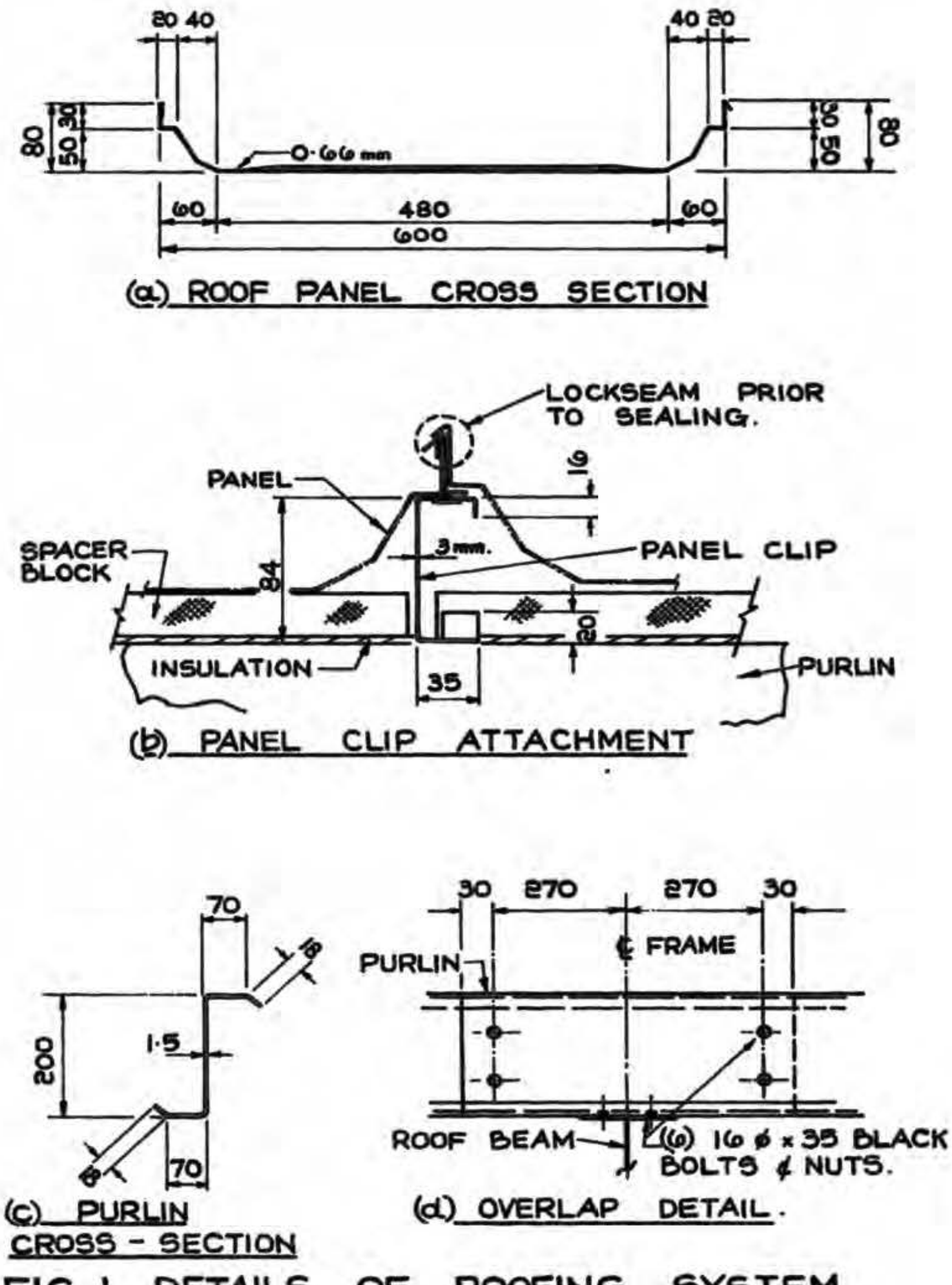


FIG 1. DETAILS OF ROOFING SYSTEM.

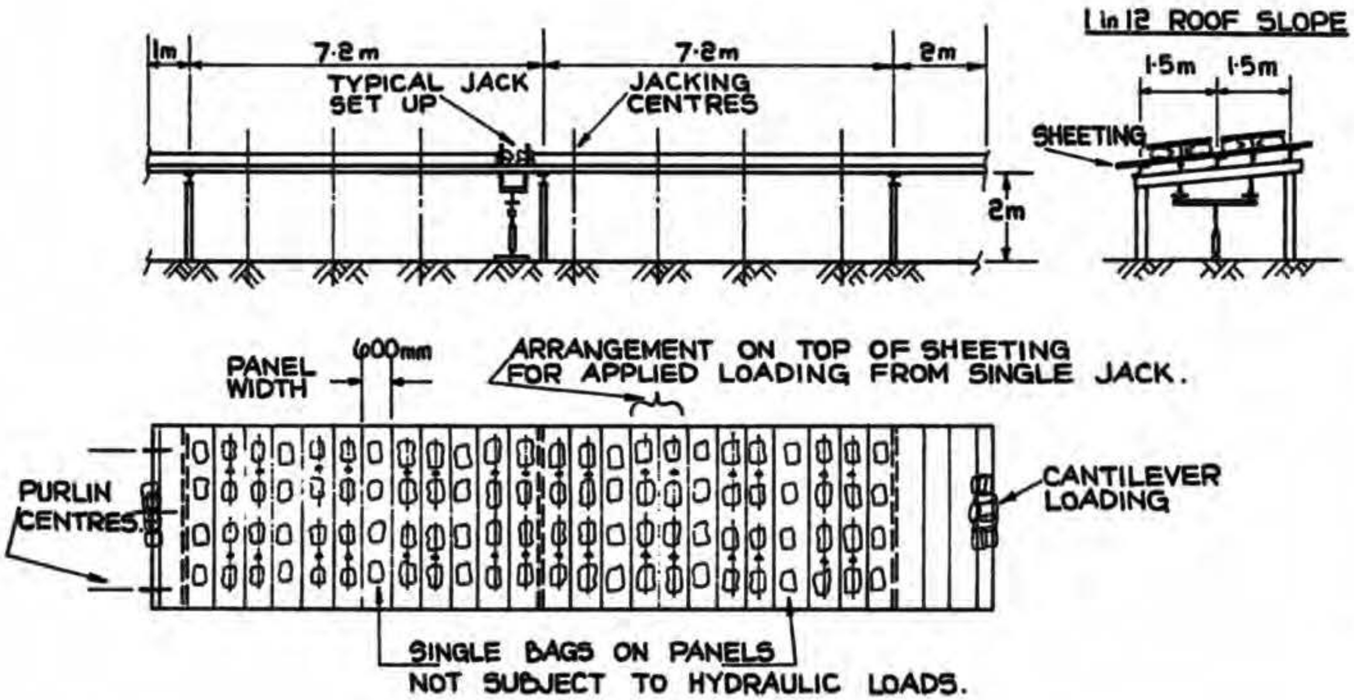


FIG 2. TEST RIG AND LOADING ARRANGEMENT (LIVE LOAD TEST)

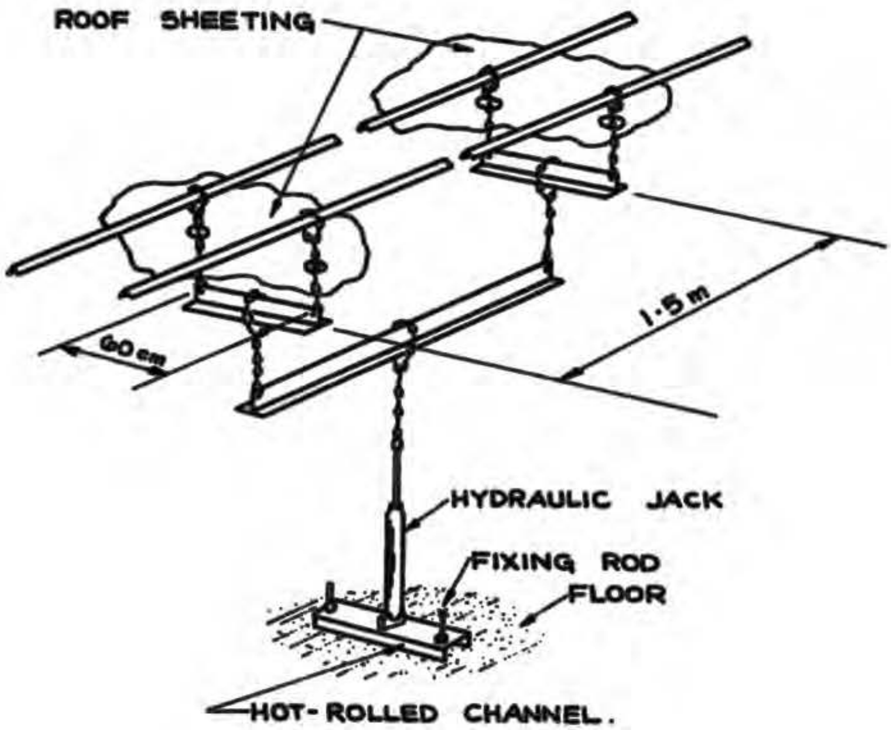


FIG 3 HYDRAULIC JACK / SPREADER BEAM ARRANGEMENT

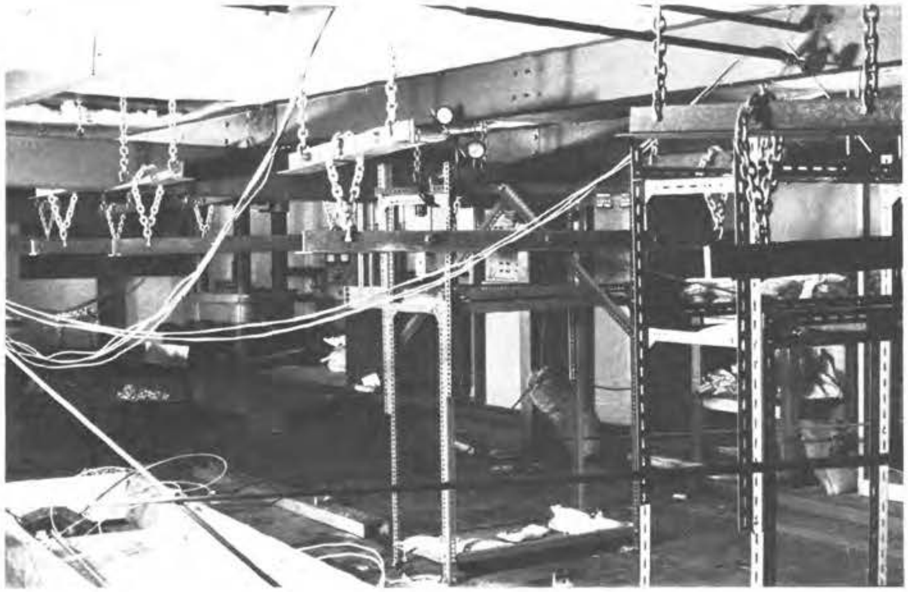


FIG. 4. UNDERSIDE VIEW OF TEST ROOF (LIVE LOAD TEST)

FIG. 5. TOP VIEW OF TEST ROOF (LIVE LOAD TEST)

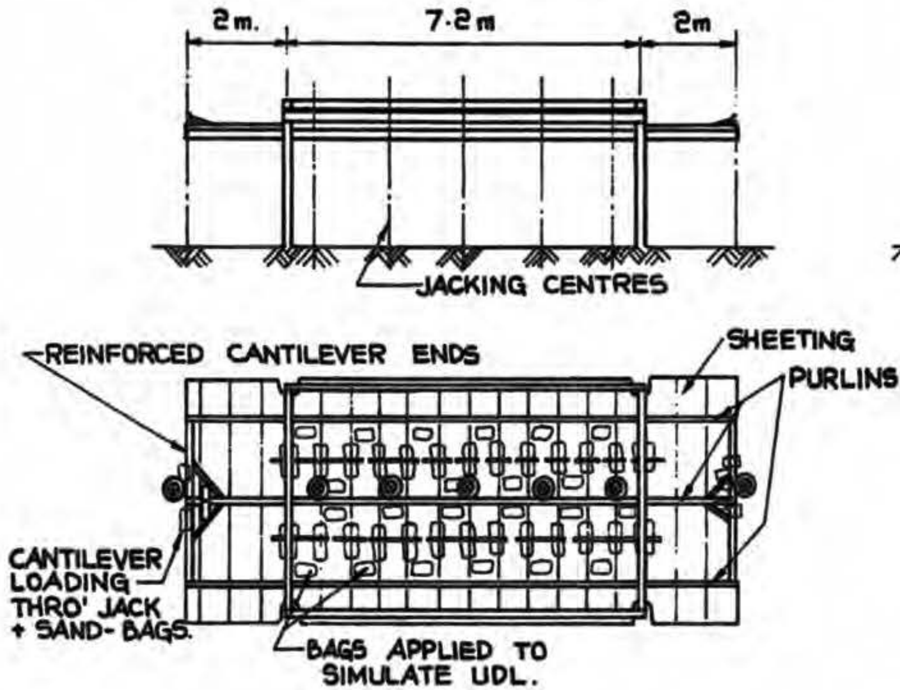


FIG 6 TEST RIG AND LOADING ARRANGEMENT (WIND UPLIFT TEST)

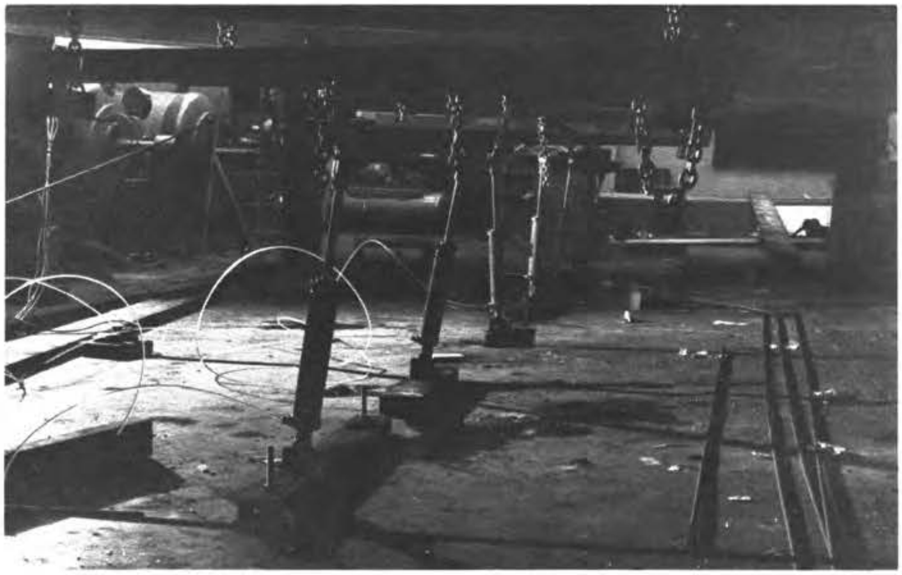


FIG. 7. VIEW UNDER TEST ROOF (WIND UPLIFT TEST)

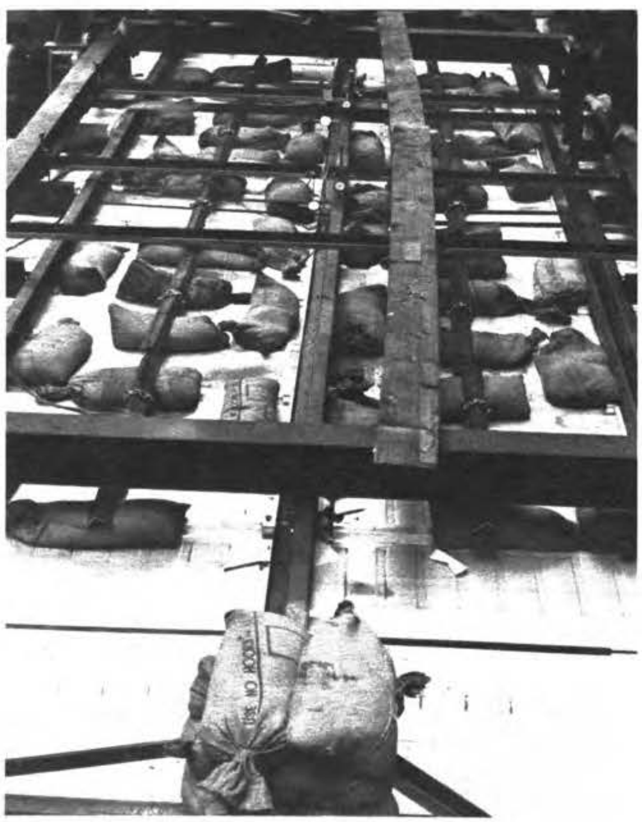
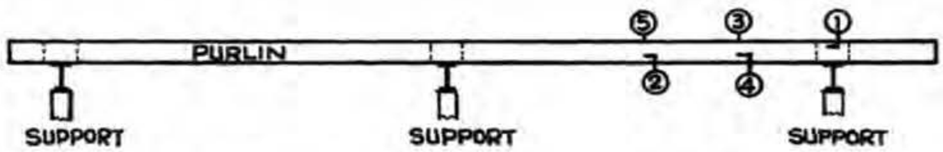


FIG. 8. TOP VIEW OF TEST ROOF (WIND UPLIFT TEST)



DEFLECTION GAUGE POSITIONS (GAUGES 1, 2, 4 HORIZONTAL 3,5 VERTICAL)

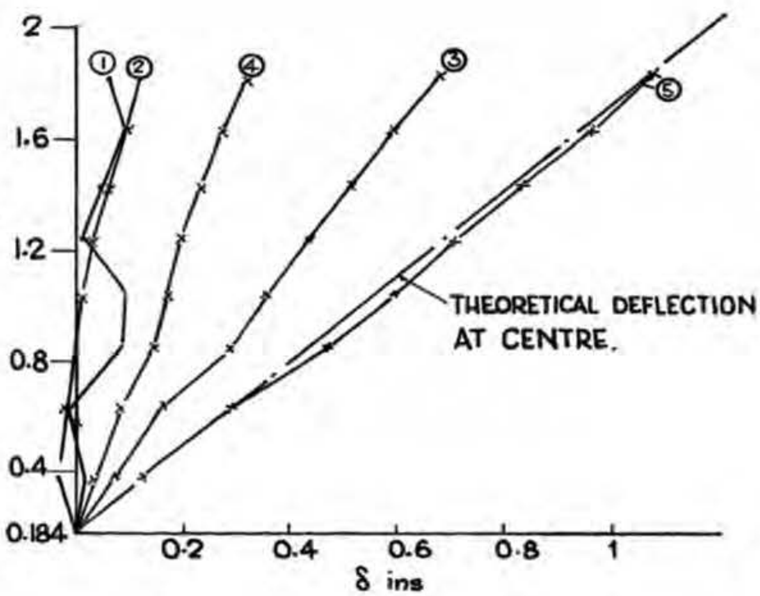


FIG. 9. CENTRAL PURLIN DEFLECTIONS (LIVE LOAD)

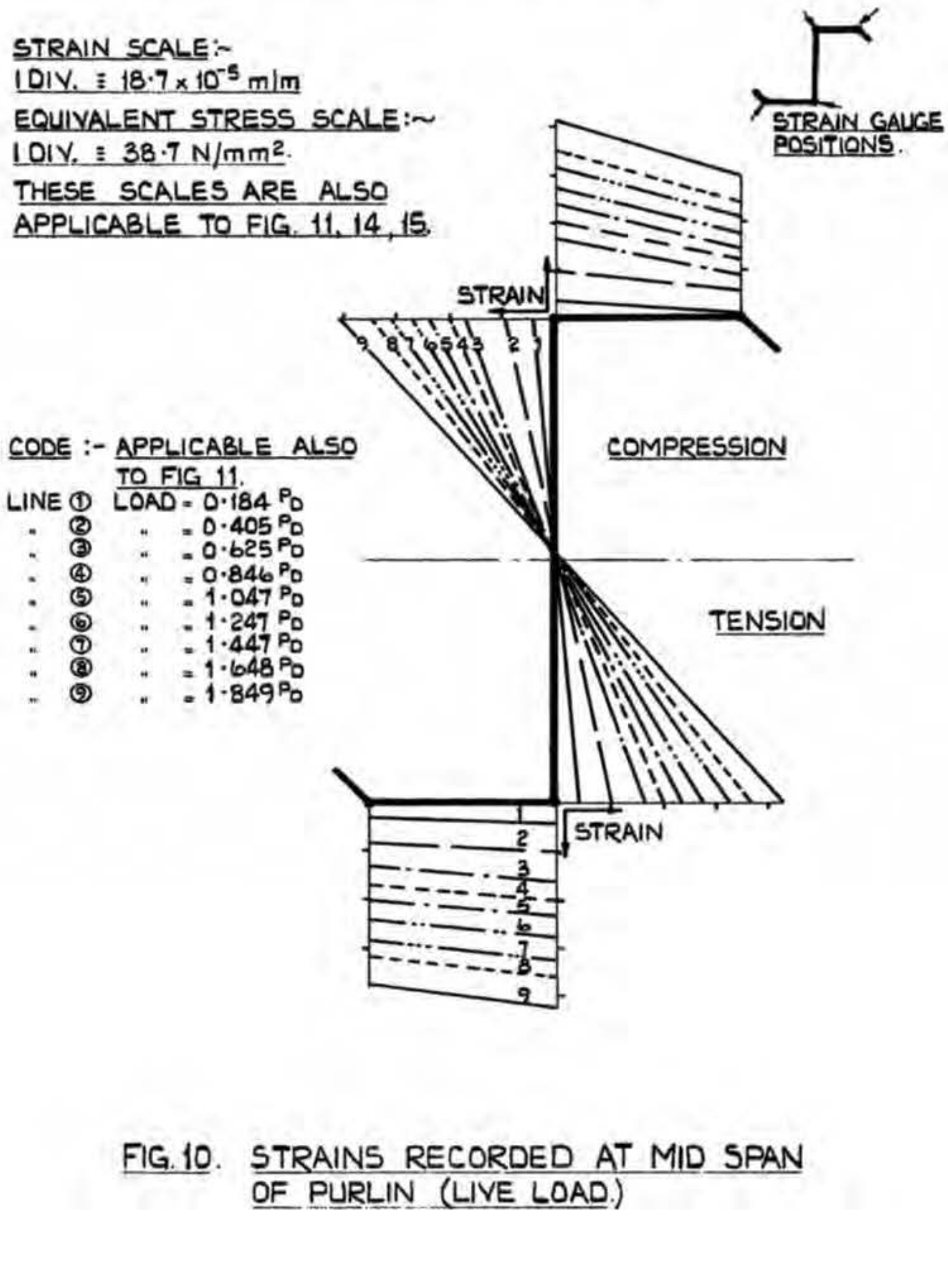


FIG. 10. STRAINS RECORDED AT MID SPAN OF PURLIN (LIVE LOAD.)

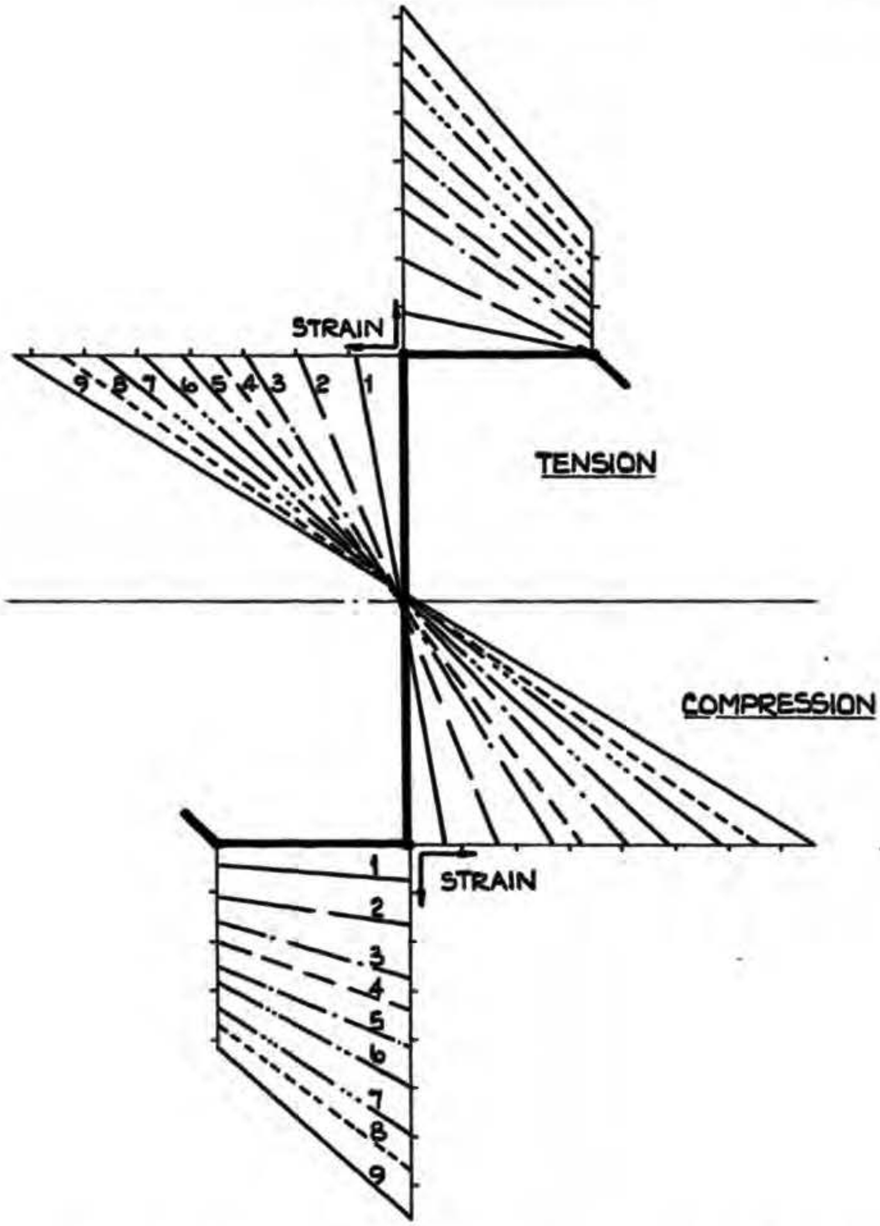


FIG. 11. STRAINS RECORDED AT LAP END OVER CENTRAL SUPPORT (LIVE LOAD)

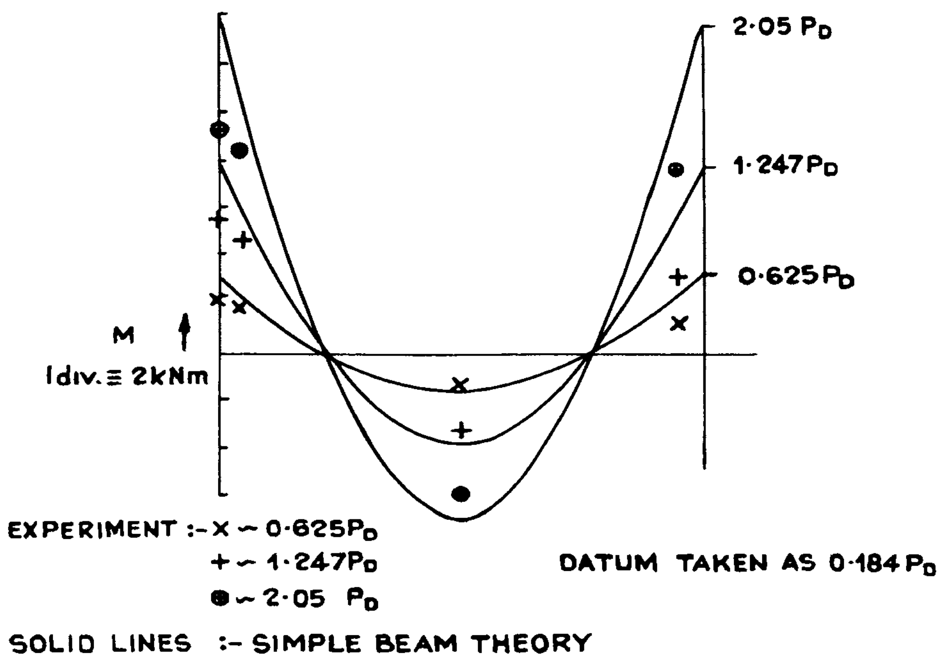


FIG. 12. BENDING MOMENT DISTRIBUTION ON PURLIN

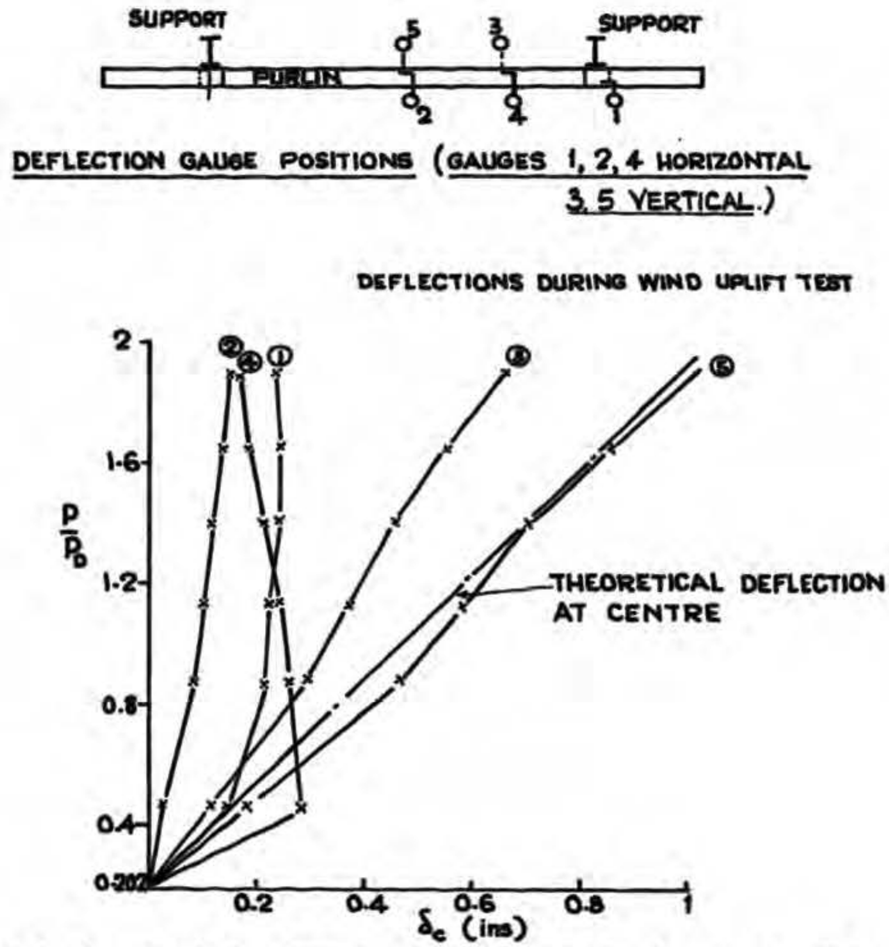


FIG. 13. CENTRAL PURLIN DEFLECTIONS (WIND LOAD)

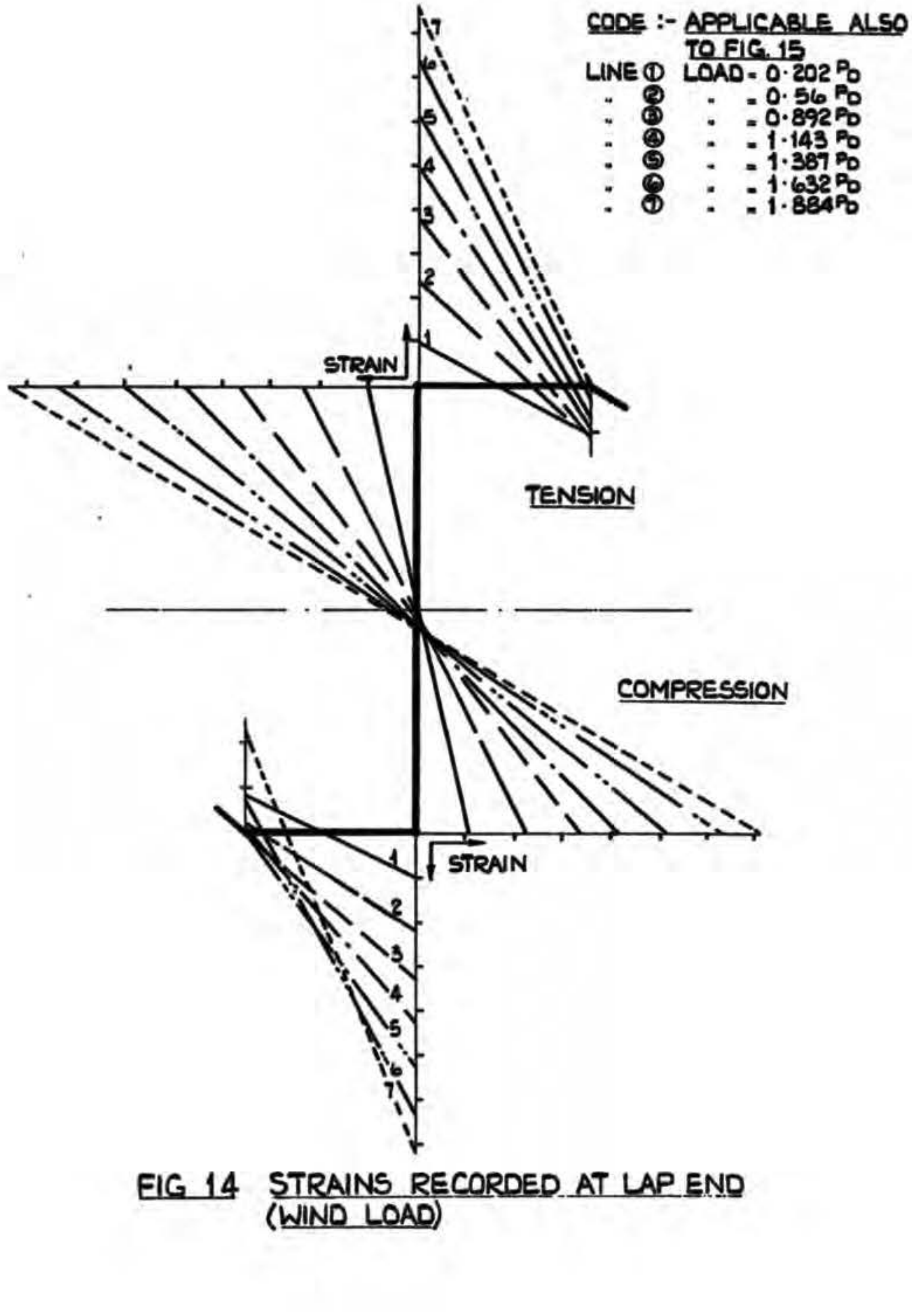


FIG 14 STRAINS RECORDED AT LAP END (WIND LOAD)

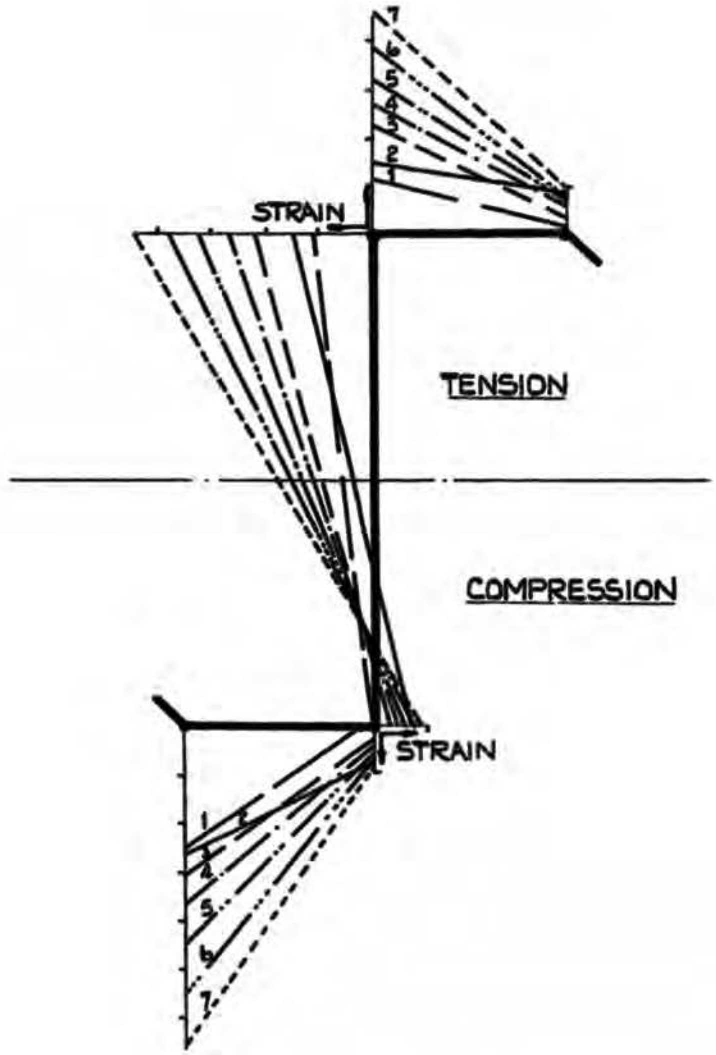


FIG. 15. STRAINS RECORDED AT MID SPAN OF PURLIN (WIND LOAD)

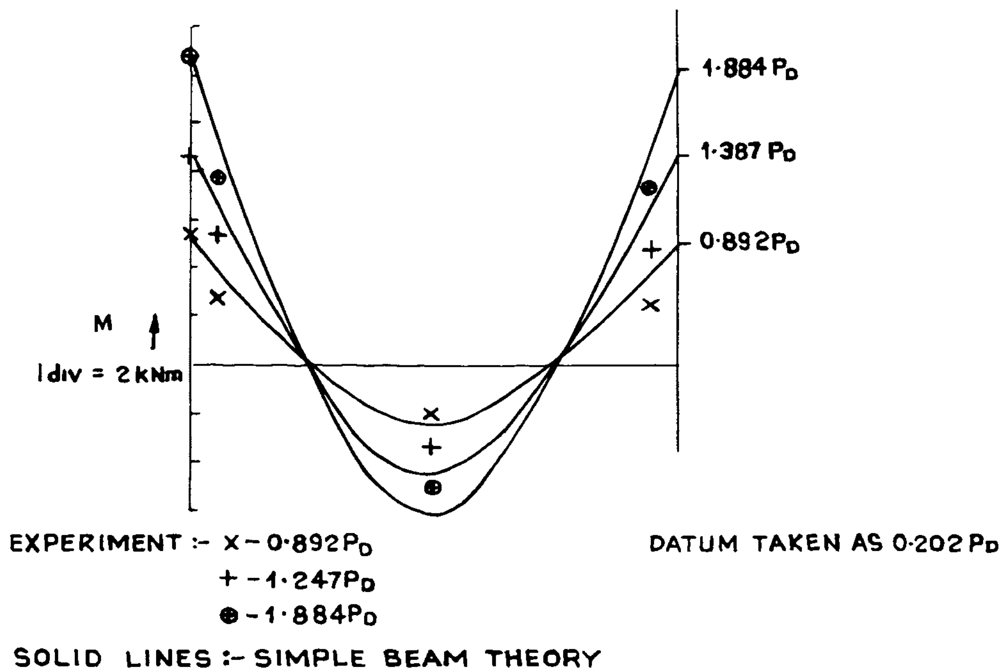


FIG. 16. MOMENT DISTRIBUTION ALONG CENTRAL PURLIN (WIND LOAD)

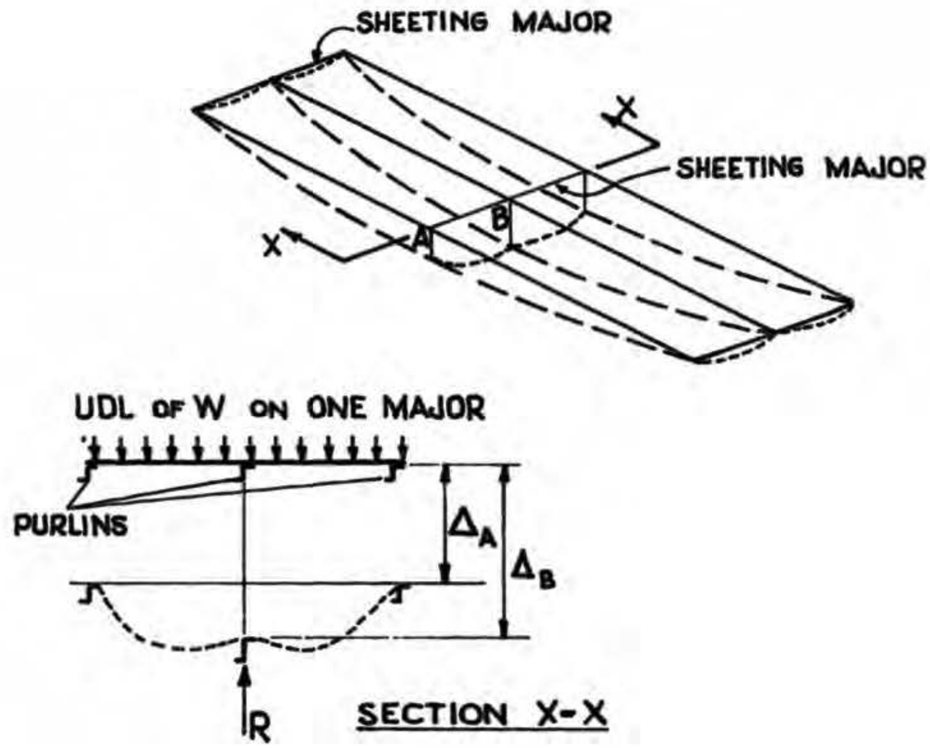

SHEETING MAJOR
X
SHEETING MAJOR
B
X
A
UDL OF W ON ONE MAJOR
PURLINS
Δ_A
Δ_B
R
SECTION X-X

FIG A.1.

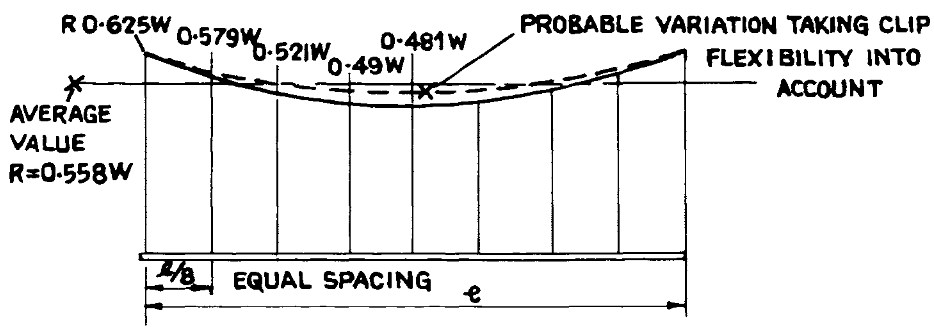

R 0·625W
0·579W
0·521W
0·49W
0·481W
PROBABLE VARIATION TAKING CLIP FLEXIBILITY INTO ACCOUNT
AVERAGE VALUE R=0·558W
ℓ/8
EQUAL SPACING
ℓ

FIG. A.2