Strains in a steel portal frame structure

R. SHEPHERD

Dr W. M. El-Dakhakhni, Assiut University, Egypt, UAR

The Author gives the impression that he is uncertain as to the manner in which the roof sheeting influences the behaviour of the steel frame, since the following points were not dealt with.

46. The location of the test portal is not specified with respect to either its distance to the end gables or the centre of the shed. It has been established^{9,10} that identical frames in a clad shed behave differently because the restraint afforded to them by the roof sheeting varies along the shed, being a maximum at the frame next to the end gable and a minimum at the central one.

47. Fig. 1 shows no details of the end gables and no further reference is made to it in the text. The stiffening of roof sheeting depends, above all, on whether the end gables are braced in their own plane. Thus, if the end portals are not braced or no structural connexions exist between them and the end walls, if any, the roof cladding will furnish no restraint whatsoever on loading all the frames simultaneously. It will still, however, have a reduced effect on loading one frame only, as in this case the spread of the loaded frame will be resisted by the roof cladding, elastically supported on the adjacent unloaded frames.

48. The statements in §§ 25 and 40 also require clarification. Normally, the load carried over from a loaded frame to the adjacent unloaded ones in the manner suggested by these statements is very small. These result from bending of continuous purlins. The main effect, however, is due to the tendency of the frame to spread under load. This tendency is resisted by the sheeting attached on the roof slopes which acts as two deep plate girders supported on the adjacent frames. Consequently, an adjacent frame is subjected to pulls at the eaves giving rise to a linearly varying bending moment along its members.

49. To produce a good comparison and help in drawing conclusions, it would have been preferable to present bending moments, rather than stresses, derived from the measured strains. This has been done using the strain values recorded by the Author and the properties of the steel sections of the frame members. It was noticed in the D.L. case of loading (Table 1) that in spite of the partial base fixity, the measured moments at the eaves and along the rafters were about 20% higher than their corresponding theoretical values. This could be the result either of underestimating the D.L. on which the theoretical analysis was made, or of an error in calculating the maximum fibre bending stress from the measured strains, in which case all the measured moments for various cases of loading are likely to be overestimated; this possibility would account for the summations of the measured strains exceeding the theoretical maximum strains in Table 4.

Dr E. R. Bryan, University of Manchester

I am glad to see that another opportunity has been taken to test a full-sized building, so making more information available to engineers on the actual behaviour of complete structures as distinct from the assumed behaviour for the purposes of analysis. If the information is to be of general use, however, it is important that great attention should be given to the deductions drawn from the results so that the conclusions are not swayed by the particular idiosyncrasies of the building under test.

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51. In § 30 it is pointed out that the concrete dwarf wall appeared to afford some horizontal restraint to the legs at about 13 ft below the knee. It must therefore be emphasized that no general deductions can be made about the measured strains below this level, i.e. at points A, M, Q and L, except to say that they should be less than the theoretical values. There were similar results in the tests described in ref. 8 when a floor slab virtually encastered a nominally pinned base.

52. The roof slope of the building under test was very flat and so it could not be expected that the roof decking would contribute substantially to the stiffness of the frames under vertical load, though it would help considerably under a horizontal wind load or crane surge. Nevertheless, it appears from Tables 2 and 4 that the contribution was by no means negligible. The roof decking behaves rather like the web of a deep horizontal plate girder, stretching from gable to gable and tending to prevent movement at the tops of the columns. It would have been of considerable help if other tables had been prepared showing a comparison of measured strains with those calculated assuming (a) feet pinned 13 ft below the knee, and (b) feet fixed 13 ft below the knee.

53. With regard to measured stresses, § 18 gives a conversion factor of 9/7 in order to obtain the stress at the outer face of the joist section. Presumably this applies to bending about the major axis and implies that the distance from the neutral axis to the internal face of the flange was only 7 in. Also, in Table 1, the direct compressive stresses at sections, A, M, Q and L give an average value of about 1.3 tons/sq. in. leading to an axial dead load on the leg of about 21 tons. In tests I have carried out it was found that measurements of axial stress usually gave unreliable results. Would Mr Shepherd care to comment further on these two points?

54. The agreement between measured and calculated strains shown in Table 4 and commented upon in §§ 33 and 34 seems to be most satisfactory, but again it should be noted that the results apply only to points on the frame at eaves level and above.

55. The comment made in § 36 is at variance with the results of other tests made on clad buildings.¹⁰ The carry-over effect mentioned is only part of the overall stiffening effect of cladding, and while the effect is particularly pronounced when only one frame is loaded, yet it is usually of considerable importance when all frames are loaded.

Mr R. Shepherd, Canterbury University New Zealand

56. Dr Bryan is entirely justified in stressing the need for care when formulating general conclusions regarding structural behaviour from records obtained on a particular building. I would fully agree with Dr Bryan's comment regarding the limitation on deductions which may be made from the strains measured in the lower third of each column.

57. Comparisons between the measured stresses and strains and the corresponding values calculated assuming the column feet to be pinned 13 ft below the knee and fixed 13 ft below the knee are presented in Tables 9 and 10 respectively. Good agreement is evident for most of the positions listed but it should be noted that the results for gauges A, M, Q and L are not tabulated. My own experience confirms Dr Bryan's, in that it appears very little reliance can be placed on the axial strain measurements which were made on the tested portal.

58. The conversion factor of 9/7 used in § 18 does apply to bending about the major axis. The vibrating wire in each of the measuring gauges was 7 in. from the neutral axis of the section.

59. It is evident that qualification of § 36 is necessary. It was not clear that the original comment in fact applied to the particular type of portal frame structure on which the strains were recorded. Certainly economies may be achieved by taking into account the stiffening effect of the roofing on pitched roof portal frames, providing that effective bracing exists in the plane of the portals.

Table 9. Maximum bending stresses and strains resulting from 9070 lb apex load

Position	Experimental maximum stress lb/sq. in.	Theoretical maximum stress lb/sq. in.	Experimental maximum strain $\times 10^{-6}$	Theoretical maximum strain $\times 10^{-6}$	Experimental strain
					Theoretical strain
C	-5300	-6500	-182	-224	0.81
K	-4860	-6500	-168	-224	0.75
P	-4550	-6500	-157	-224	0.70
D	-5560	-6800	-192	-233	0.83
J	-5520	-6800	-191	-233	0.82
E	+1710	+1400	+59	+48	1.23
H	+1650	+1400	+57	+48	1.19
F	+7100	+7200	+245	+250	0.98
G	+6600	+7200	+225	+250	0.90

(Theoretical values calculated assuming portal feet pinned 13 ft below knee.)

Table 10. Maximum bending stresses and strains resulting from 9070 lb apex load

Position	Experimental maximum stress lb/sq. in.	Theoretical maximum stress lb/sq. in.	Experimental maximum strain $\times 10^{-6}$	Theoretical maximum strain $\times 10^{-6}$	Experimental strain
					Theoretical strain
C	-5300	-4500	-182	-155	1.17
K	-4860	-4500	-168	-155	1.09
P	-4550	-4500	-157	-155	1.01
D	-5560	-5900	-192	-203	0.95
J	-5520	-5900	-191	-203	0.94
E	+1710	+300	+59	+10	5.90*
H	+1650	+300	+57	+10	5.70*
F	+7100	+5600	+245	+192	1.27
G	+6600	+5600	+225	+192	1.17

(Theoretical values calculated assuming portal feet fixed 13 ft below knee.)

* Discrepancy accentuated by the position of the theoretical inflexion point in the rafter.

60. Dr El-Dakhkhni's contribution is particularly pertinent in that his summary of the manner in which roof sheeting can influence the behaviour of the portal frames may be used to clarify the apparent discrepancy between the comment in § 36 and the results of the valuable work undertaken by Dr Bryan and Dr El-Dakhkhni.¹⁰

61. The omission of details of the end walls and of the location of the test portal is regretted. The main test portal was in fact 60 ft from one end of the building, as initially erected. The end filler walls consisted of precast concrete panels bolted to steel girts welded to the end portals. Subsequently the building was extended by four bays at the end nearest the test portal, the original gable wall being relocated and replaced by a light timber partition.

62. I do not consider that the gable walls would provide sufficient in-plane bracing to allow the development of the behaviour pattern outlined by Dr El-Dakhkhni. The symmetry of the load distribution illustrated in Fig. 5 supports this

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contention. If the end gable walls did provide sufficient in-plane bracing to resist the spreading of the portals, consistently smaller strains would probably have been recorded when the 9070 lb load was applied to the portals nearer the gable than when the load was applied to the frames nearer the centre of the building.

63. Since the particular pitched roof portal frame structure which was the subject of the report did not appear to be capable of developing any effective end wall in-place bracing, it seems that use of roof cladding stiffness considerations would not have led to a significantly more economical design.

References

9. EL-DAKHAKHNI W. M. Ph.D. thesis, University of Manchester, 1963.
10. BRYAN E. R. and EL-DAKHAKHNI W. M. Behaviour of sheeted portal frame sheds: theory and experiments. *Proc. Instn civ. Engrs*, 1964, **29** (Dec.) 743-778.