Paper No. 6495

Structural problems in the use of cold-formed steel sections†

by

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Discussion

Professor Sir Alfred Pugsley (University of Bristol) wished to begin by making the general observation that when new materials were introduced into civil engineering they were not only of immediate benefit to the profession but had also a much wider and in the long run more important effect: they gave rise to new structural problems, the solution of which caused a more general advancement of knowledge. Most new materials had had that effect, and this was very well exemplified by the Author's work. The advancement in this case had initially been of a detailed analytical character, exemplified by the Author's analysis of local buckling. But it had since led to a deepening of the understanding of the behaviour of structures generally; and the relevance of the local buckling problem to the behaviour of structures and structural components in general was illustrated in the Paper.

57. There were two details in the Paper on which he wished to comment. There was an interesting little section on the tendency for a tension member, even when axially loaded in a particular distributed fashion, to twist when tension was applied. Did the Author regard this as important in practice? These light cold-rolled sections had very low torsional stiffnesses; was it not very easy to suppress a twist of that sort by local connexions if it were desired to do so?

58. In another section of the Paper reference was made to the economy and efficiency of the use of cold-rolled sections and this was nicely illustrated by some curves of the permissible stresses for struts in relation to the structural loading coefficient (the end load divided by the length squared). In this graph, Fig. 22, the cross-over indicated the range over which cold-rolled sections as struts were particularly efficient. To avoid using 10^{-3} and the like, if one multiplied by 2,240 to bring the figures to lb/sq. in. it would be found that the cross-over occurred for a coefficient between 2 and 4 lb/sq. in. That was a pointer—and no doubt the Author meant it as such—to the sphere in which sections of this sort were specially valuable: lightly loaded structures where the members were relatively long. The range of structure loading parameter in civil engineering practice was much wider and went up to 10 or 15 lb/sq. in., and at the upper end no one would wish to use cold-rolled sections; but that was no unusual disadvantage because all materials and all sections had such limited ranges of efficiency.

59. Sir Alfred had hoped that the Author would indicate more than he had in fact done the work to which he looked in the future. It might be that he felt that local buckling had had its day. He had done his work so well that he might not want to touch it up further, but what did he think was the next step? One possible step came to mind on reading the introduction to the Paper, in which it was pointed out that the process of forming cold-rolled sections cold-worked the material very heavily where the plate was bent most, at the "corners", so that the local hardness went up. One wondered whether or not more work should be done on the fatigue of cold-rolled steel

† Proc. Instn civ. Engrs, vol. 20, pp. 232–258 (Oct. 1961).

structures, to include for fundamental reasons some fatigue tests on isolated members so loaded that their "corners" were in longitudinal shear. Was any such work planned?

Mr L. R. Creasy (Superintending Engineer, Ministry of Works) said that the Author had very ably indicated the complexities of design in using these thin cold-formed sections and had indicated the background of research which had gone so far towards solving the problems involved. The list of references at the end of the Paper would be of particular value to those who sought a wider review of the subject.

61. These thin sections presented a very useful alternative form of construction in certain spheres as compared with the conventional hot-rolled fabrication. Their outstanding characteristic was, of course, the saving in unit weight of metal achieved by a more efficient distribution of material. For light trusses of 20-ft span it was not very difficult to achieve savings approaching 60% of the metal content as compared with the alternative hot-rolled forms. As the span increased the saving would become less and perhaps for a span of 50 ft might be nearer 40%. The unit fabricating costs would be higher, and "break-even" point at which these thin sections became competitive with the conventional hot-rolled fabrication occurred where the saving in weight approached 50%. This would largely depend on the degree of repetition, bulk ordering, and standardization which could be achieved. The thin sections were not generally suitable for small one-off structures or those which were complicated or of large span and high stress.

62. They became particularly economic when used as secondary members in combination with hot-rolled fabrication, particularly where the designer could choose standard units available on the market, of which purlins and floor joists were outstanding examples. The maximum efficiency in many projects was achieved by a judicious blending of hot-rolled and cold-formed fabrication.

63. Much had been said about corrosion and obviously this was an important factor, but it was one to which too much difficulty ought not to be assigned. The cold-formed section must be protected from corrosion, as must any exposed steel form, with the help of a coating. It was this coating which must be protected in all cases. It was not more difficult to apply a coating of paint, metal spray, or hot-dip galvanizing to the light sections and indeed, the greater ease of handling often made it easier to provide some of the more advanced coatings. It was true that for a specific loss of metal there would be a greater encroachment on the load factor, but it was unwise to allow any loss of metal whether the section was thick or thin and for this purpose the prudent user would ensure the life of his fabrication by the maintenance of the protective coat.

64. The Author might perhaps have given a somewhat forbidding impression of the difficulties of design in these thin sections, but he would not mind it being said that the new Code of Practice, Addendum No. 1 to B.S.449, made a very great effort to simplify the problems of design, based largely on the research work of the Author and others in this field. The speaker then briefly indicated the simplified design approach as given in that Code together with illustrations of the use of combined cold-formed beams and hot-rolled columns as used in the recent rebuilding of the British Museum Library.

Mr W. Shearer Smith (Chairman, Technical Committee, Cold Rolled Sections Association) congratulated the Author on an excellent survey of the main design problems which could arise in the use of cold-formed sections as structural members. A great deal of research had been done during the last 15 years, involving thousands of specimens. Many aspects had been dealt with and others were still being vigorously investigated. Mr Creasy had made one of the points which Mr Shearer Smith had intended to make, namely, that many of the problems outlined in the Paper might appear at first sight to be complicated, especially from the designer's point of view; but fortunately, thanks to the work of the Author, Mr Creasy, and others, this complication largely disappeared by reason of Addendum No. 1 to B.S. 449, published early in 1961.

66. While the research teams had been busy, the industry had not been altogether

idle. Fig. 47 showed a warehouse consisting entirely of cold-formed sections, covering an area of about 750,000 sq. ft. It was fabricated entirely of these sections by spot welding and arc welding with site bolted connexions. The frames had a span of 49 ft at 20 ft spacing. Fig. 48 showed the use of cold-formed sections in conjunction with hot-rolled sections. The roof trusses and purlins were cold-formed. The combination was ideal for a building of this type. Fig. 49 illustrated the large doors 33 ft high for a B.O.A.C. building at London Airport. Fig. 50 showed a skeleton for a double-decker bus, using COR-TEN steel thus providing increased corrosion resistance together with high-tensile strength. Fig. 51 had a distinct civil engineering flavour, being steel shuttering. The channel panels were 12 in. wide.

Dr R. M. Kenedi (The Royal College of Science and Technology, Glasgow) said that research on cold-forming had now been going on for something like 15 years. The Paper outlined the present-day problems, but it was interesting to reflect that 15 years ago while so many problems had not been solved there had not been so many problems appearing either. Research had a way of solving problems and also producing problems, and like John Brown's soul "kept marching on".

68. Reference was made in the Paper to structural connexions, on which they had been conducting research in Glasgow for the past two years, as one of the avenues now being explored more intensively than before. Dr Kenedi had a few slides which showed the sort of difficulties which were being encountered and the surprising results which were being obtained. Fig. 27 of the Paper was a photograph showing, basically speaking, two members with a couple of gusset plates welded to them back and front. The theoretical problem (Fig. 28) was that of local instability of a plate with a discontinuous loading along one edge. To put it politely, the theoretical solution of this problem as yet escaped them, as it escaped those mathematicians that they had consulted; but they had managed to obtain what was a semi-empirical solution using the same sort of basic variables as in the uniformly-loaded local instability case.

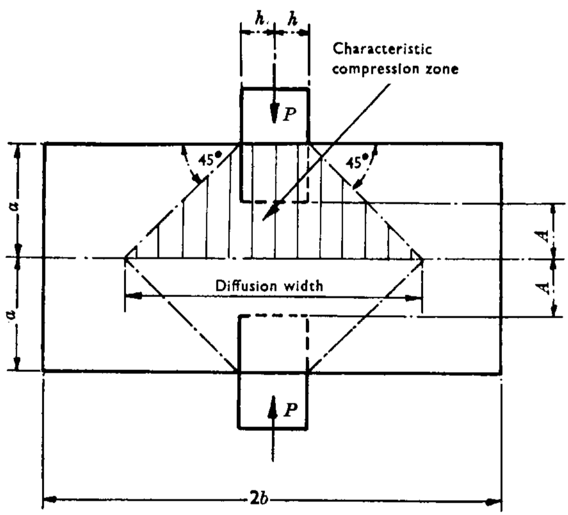


FIG. 28.—BASIC PROBLEM

* Figs 47–61 are photographs and are placed between pp. 280–281.

69. From this point they had gone on to the investigation of actual joints, using a series of frames (Fig. 29) built for them by the Cold Rolled Sections Association.

70. One of the reasons incidentally, why cold-formed structural research produced results was that there was very close co-operation indeed with the industry, who kept the feet of the academic workers firmly on the ground.

71. The design of this type of frame was a little too ingenious; it incorporated a variety of joints—a two-member joint, two kinds of three-member joints, a five-member joint and so forth. Loading the frame in different ways a variety of force conditions could be induced in the joints.

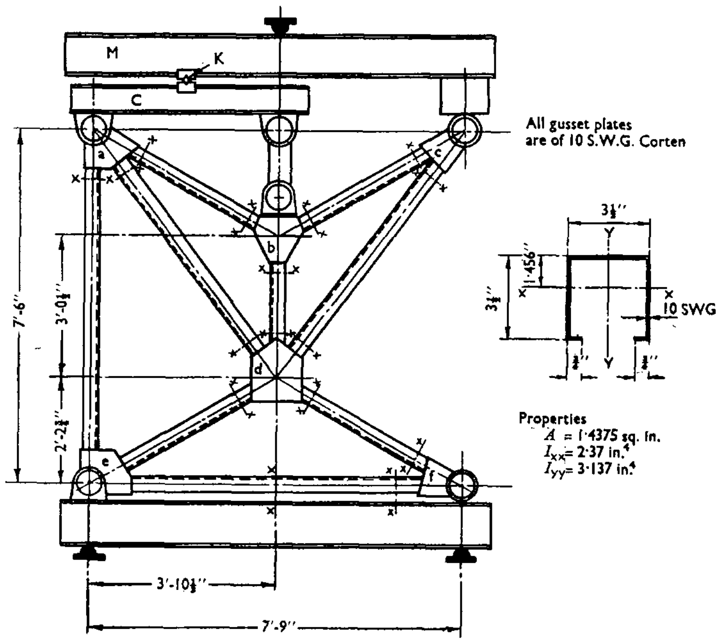


FIG. 29.—TEST FRAMES

72. Fig. 30 gave an idea of how the various load conditions were induced. There was great flexibility. Experimental readings were taken so that they could evaluate the load in each particular member. The frames were loaded until failure was obtained due to buckling of the gusset plates of the various joints in turn. Fig. 31 showed a feature which might seem obvious, but it was shown for the first time experimentally. The channels were welded to the gusset plates along their flanges, and, as was to be expected, on coming near the gusset plates the average stress in the flanges was relatively higher, the flanges being the connected components of the sections, than in the webs or in the lips. This was a feature which they proposed to investigate further, because it seemed to them that in frames of this type there might be an undesirable building up of the local compressive stress.

73. Fig. 32 showed the modes of failure in the gusset plates. While they tried to control the failure mode as far as they could, due to the irregularities present they had to take what the frames gave them. They had therefore investigated the mode very carefully. With the joint *f* shown there had been no relative movement of the two ends of the joint and they had obtained a buckled form expressible as two halfwaves. In

the next case shown—joint *c*—one end moved relative to the other end, so that effectively one half wave was obtained. In the case of joint *a* one end again moved and they again got a half wave. For joint *b* there was a fixed end effect with two half waves. They had been able, therefore, to get the typical modes of failure in the various joints, and the next step had been to try to connect this in some way with the two-member-in-line joint behaviour.

74. Fig. 28 showed a shaded area which was considered to be characteristic of the particular two-member joint. In the joints shown in Fig. 33 they looked for similar characteristic zones which could be considered to govern the gusset-plate behaviour.

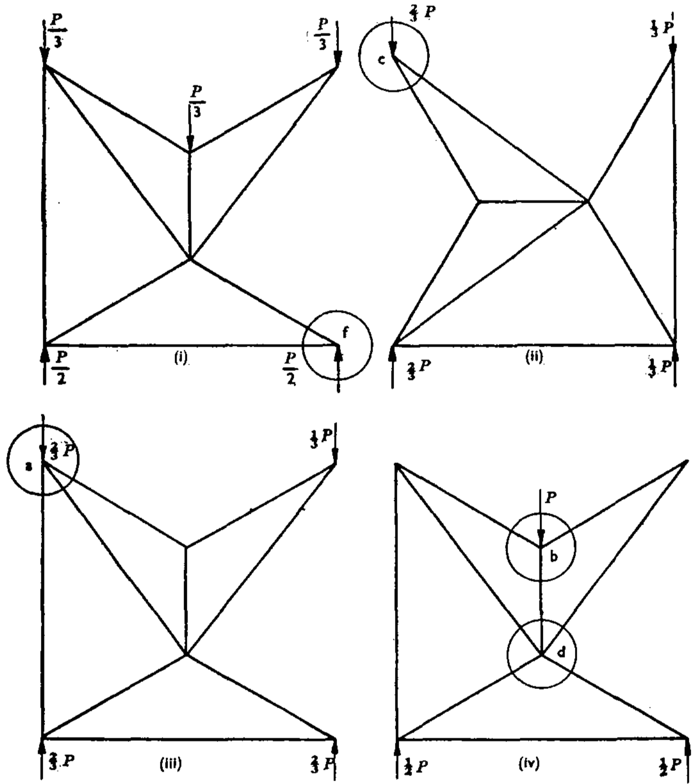


FIG. 30.—TEST LOADS

In most of these cases such a characteristic compression zone could be located: a part of the gusset plate sensibly in uniform compression. To this they had boldly applied methods derived from the two-member-in-line joint investigations to assess the collapse load of the multi-member joints. Fig. 34 showed the result. The full line was the semi-empirical curve derived from the two-member-in-line joints; the specific points were experimentally obtained for the joints shown in Fig. 33. They had found, therefore, that the compression zone concept was rational and that they would be able to predict with reasonable accuracy the collapse load of a fairly irregular-looking joint, with one proviso, that the joint must be such that its characteristic compression zone was definable.

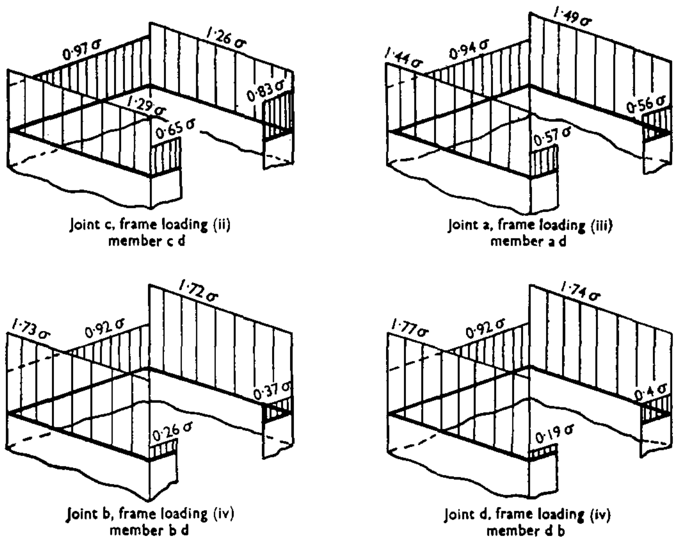


FIG. 31.—AVERAGE STRESS IN MEMBERS ADJACENT TO JOINTS

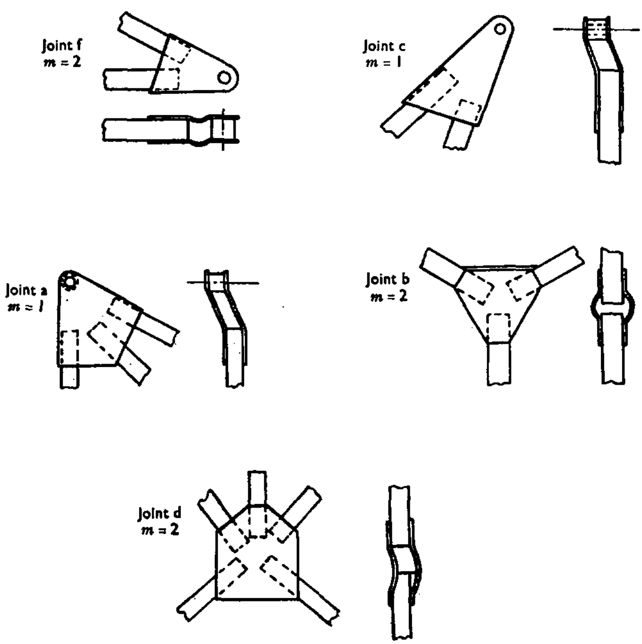


FIG. 32.—FAILURE MODES OF GUSSET PLATES

Mr E. Griffin (Chief Engineer, Metal Sections Ltd), who expressed his admiration of the Author's excellent Paper, said that his part in the discussion was merely to explain one or two points mentioned by the Author, particularly with regard to the cold-forming and cold-working of the sections, a point mentioned by Sir Alfred Pugsley, and later to try to explain the actual effect of cold-forming.

76. Fig. 52 showed the type of material coming into their works. It was in widths of 36–40 in., in coils 5 tons in weight and in thickness up to $\frac{3}{16}$ in. In the near future they would go up to 0·25 in., but at the moment they were manufacturing to a maximum

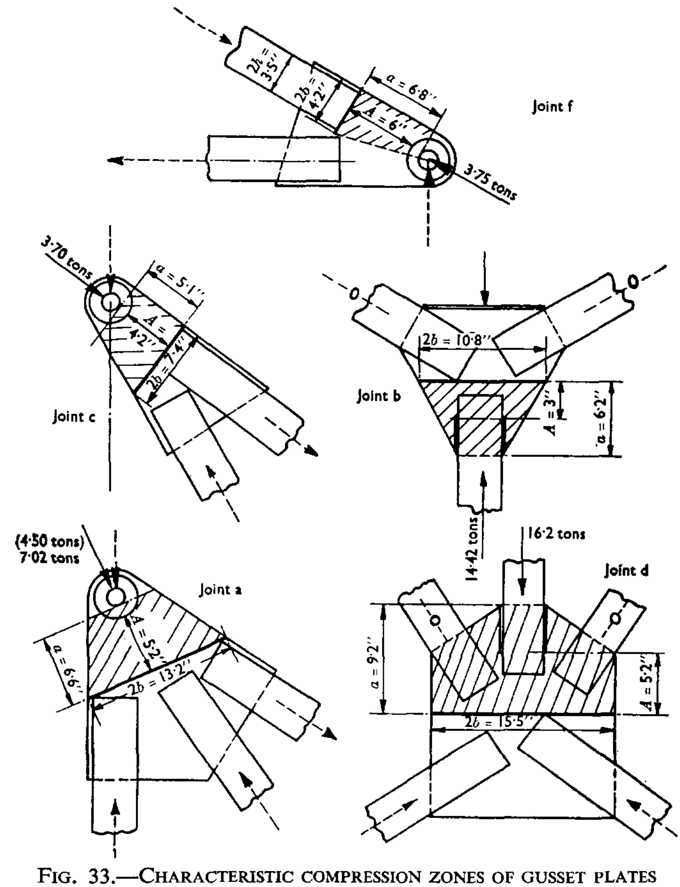


FIG. 33.—CHARACTERISTIC COMPRESSION ZONES OF GUSSET PLATES

thickness of $\frac{3}{16}$ in. The wide strip was then transferred to the slitting machines (Fig. 53) and re-coiled on the re-coiling stand at the far end. It was then ready for transfer to the cold-roll forming machine. The type of machine shown in Fig. 54 is capable of forming strip up to 22 in. in width and up to $\frac{3}{16}$ in. in thickness. The photograph showed the type of forming operation carried out on the strip as it progressed continuously across the machine, coming out as a finished section at the end.

77. During cold-roll forming it was also possible to incorporate many other operations, such as pre-piercing, bending, automatic cutting, indenting and many others of a continuous nature.

78. Small sections very limited in quantity and strip which was wider than what could be accommodated on the forming machines could be formed on press brakes (Fig. 55). The disadvantage of the type of machine illustrated was that it was limited in length and produced one form at a time, so that some of the complicated sections used with joists would be very difficult to produce on this machine. Fig. 56 was a close-up of the press rake tool showing the type of forming. It showed the Z-type of section which had just been formed.

79. After cutting off the sections could be pierced, crimped or formed in any sort of fashion. Fig. 57 showed a press doing the piercing operation. It was used for piercing because of its great accuracy and the cleanliness of the punch when it went through the material. There were many other things that could be done, and this was just an example of what went on on a cold-rolled section after it had been rolled.

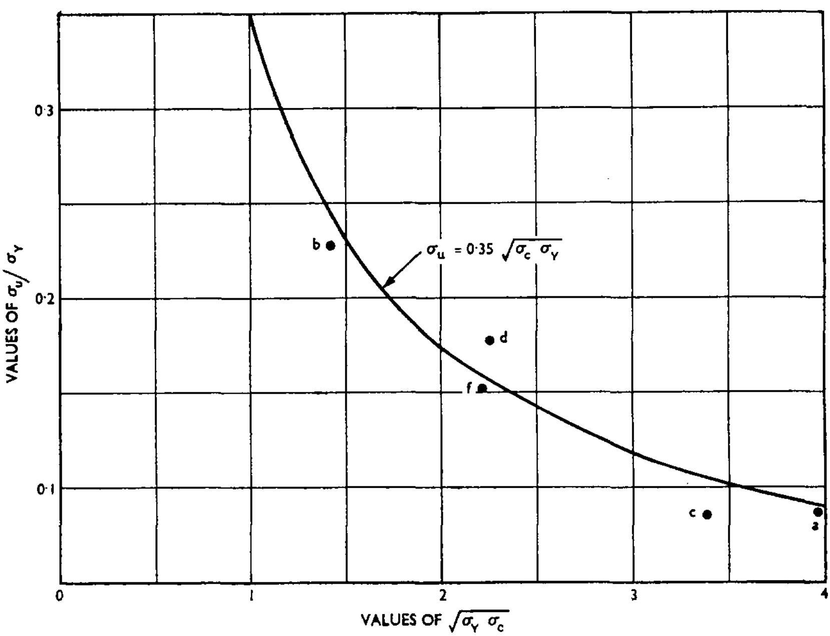


FIG. 34.—COLLAPSE STRESS VARIATION OF GUSSET PLATES

80. The line illustrations gave some idea of the effect of cold-forming on the sections themselves. Fig. 35 showed a very simple channel section with the hardness of the material taken at various points round the cross-section. There was an excellent hardness in the corners gradually reducing towards the top of the flange. Normally on a section of that size the effect of cold-rolling or cold-forming would be to increase the yield by 5 to 7%. These sections were normally formed on the inside radius. Complicated shapes involved heavier forming. The channel section shown in Fig. 36 had been produced because of the effect of cold-forming, which was not so apparent in Fig. 35. The increase in this case was nearly 15%, but that was due to the type of set-up used for this particular section. Basically the strip itself was harder than that shown in Fig. 35 and the cold-forming effect on the corners seemed to be a little greater and also the cold-forming effect on the base, but not a great deal; 15% would be as much as one would expect to get from this type of section.

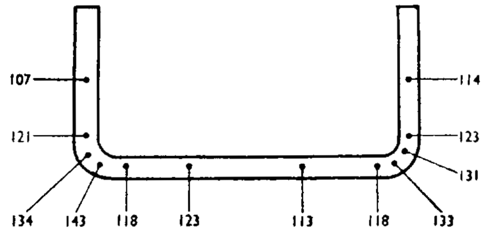


FIG. 35.—EFFECT OF COLD-FORMING ON A SIMPLE CHANNEL

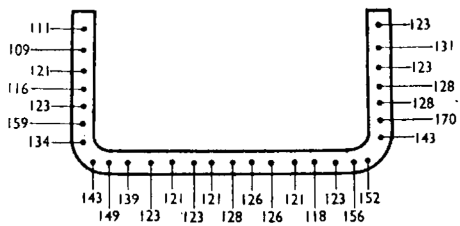


FIG. 36.—EFFECT OF COLD-FORMING ON A SIMPLE CHANNEL

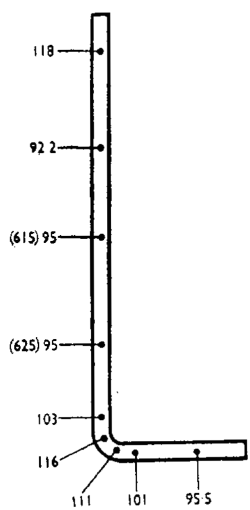


FIG. 37.—COLD-FORMED ANGLE ON PRESS BRAKE TOOLS

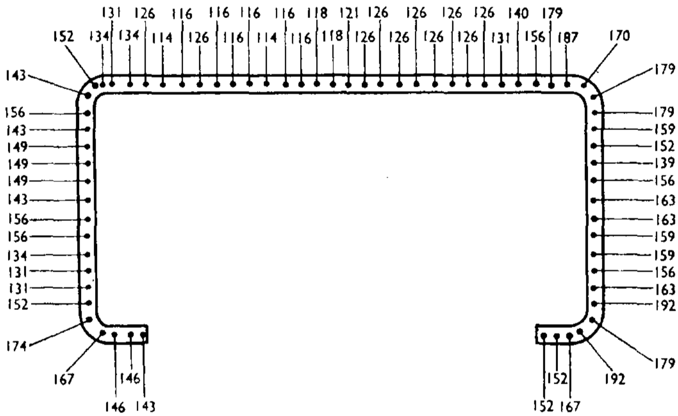


FIG. 38.—EFFECT OF COLD-FORMING ON AN INWARDLY CHANNEL

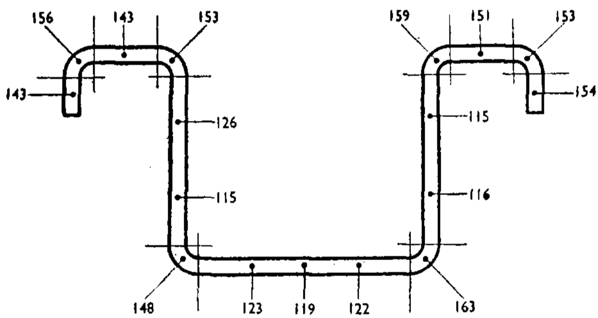


FIG. 39.—EFFECT OF COLD-FORMING ON A LIPPED TOP HAT SECTION FORMED FROM 10-GAUGE MATERIAL

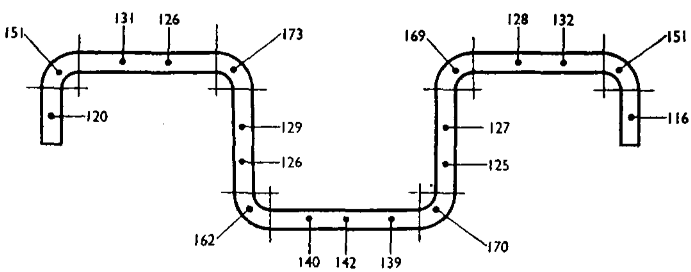


FIG. 40.—EFFECT OF COLD-FORMING ON A LIPPED TOP HAT SECTION FORMED FROM $\frac{3}{16}$ IN. MATERIAL

81. Fig. 37 showed a very simple section produced on the press brake. The cold-working effect on the corner was very much less than that shown in the other example. There was a high point, but that was due to the actual shearing of the material using a guillotine shear. The sections formed on the press brake were influenced at the corners only by work-hardening, and the overall percentage of that would be no more than 1 or 2. The section shown in Fig. 38 gave a very different picture, with heavy forming in the corners, going up to 174 D.P.N. and with figures of 152, 170 and also 179 and 182. This also indicated the accuracy of the set-up. If there was inaccuracy of the set-up, heavy forming could occur on one side of the section and not on the other. If that took place it showed that there was some inaccuracy in the set-up and this could be traced back to the production tools to find the cause; but normally, in a set-up which was reasonably good, the section shown would increase in yield by about 27% but the increase in the ultimate stress after forming the section would be only 10–15%. It was the yield of the section which was greatly increased, not the ultimate.

82. Fig. 39 showed a different type of profile, with a figure of 119 d.p.h. at the bottom and then higher figures round the cross-section which were due to cold-working. On a section of this type the increase in yield would be expected to be at least 25%. Fig. 40 showed a very similar type of section but in a much heavier material. There were concentrated areas where forming had occurred and had increased the hardness by a substantial amount. On this section, by taking the hardness around the cross-section and taking the summation of the hardness and dividing by the width across the cross-section, there was an increase of 27%.

83. Mr Creasy seemed to think that cold-rolled sections were most economic only in the shorter spans of any structural member. It might be of interest to mention, therefore, that the associates of Mr Griffin's firm in the United States of America were now designing cold-rolled-formed lattice joists up to 152 ft in span. They were doing 100 ft in this country. They were also developing cold-rolled sections in conjunction with other types of material. At the moment they were designing cold-rolled-formed joists to use in conjunction with hot-rolled structural steel joists and also universal sections. They planned in future to use cold-rolled-formed lattice joists in conjunction with concrete and hoped that there would be a great saving in multi-storey work with that type of construction.

Mr E. M. Lewis (Associate and Director of Research, W. S. Atkins & Partners) observed that those who were concerned with development work rather than research were often forced to extrapolate well ahead of what research workers were doing, or, to be more honest, were forced to make "inspired guesses". On occasions such as the present one, it seemed appropriate to record what the results of these guesses had been. From this, pointers might emerge which would indicate what could best be the next object of research.

85. Dr Chilver's paper was concerned primarily with the use of cold-rolled sections used as structural components, such as supporting beams or compression members, with b/t ratios up to 75. However, a far greater tonnage of strip was consumed in roof decks and side cladding sheets in which b/t ratios up to 250 were commonplace.

86. Fig. 41 showed the results of experimental work by the author's firm on sheet profiles over the last fifteen years. The effective width of the compression flanges calculated from the collapse load of the sheet was compared with those predicted by Winter's theory quoted as equation 37 in § 45 of the Paper. With the exception of two points, the results supported the Winter curve as a lower limit, as the points all lay above this curve.

87. The point which was circled was of interest as it was based on a test result on an aluminium sheet which was designed for use on the Abbey Works at Port Talbot some fifteen years ago and which proved to be the forerunner of a range of roof deck sheets now widely used in industry. There was available at that time only elementary theory.

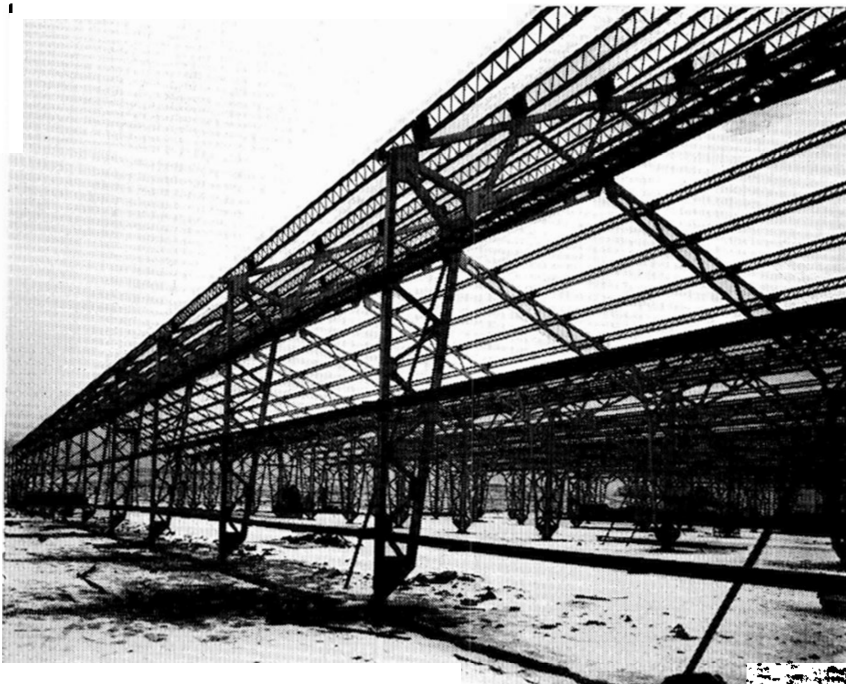


FIG. 47.—LARGE PREFABRICATED WAREHOUSE CONSISTING ENTIRELY OF COLD-FORMED SECTIONS

FIG. 48.—40-FT SPAN SINGLE STOREY FACTORY. USE OF COLD-FORMED SECTIONS IN CONJUNCTION WITH HOT-ROLLED SECTIONS

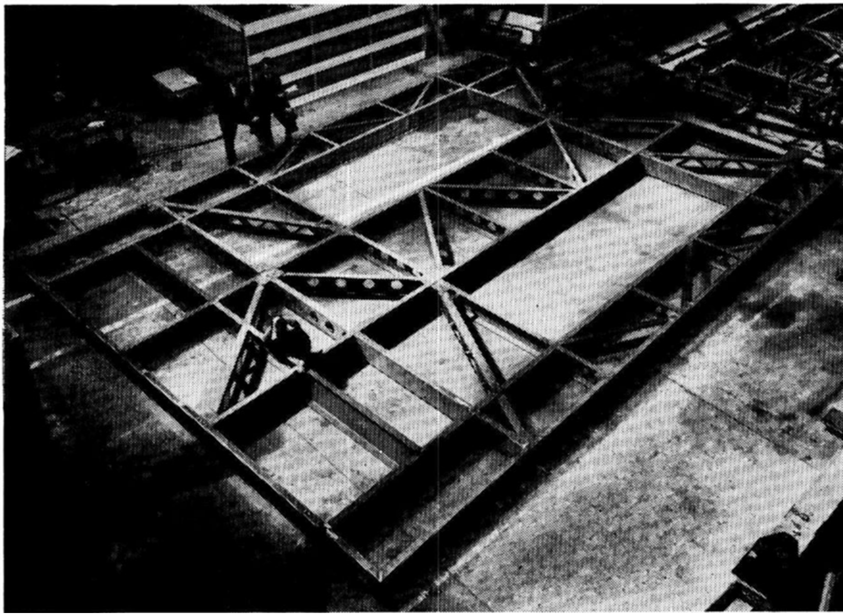


FIG. 49.—33-FT HIGH DOORS FOR B.O.A.C. BUILDING AT THE LONDON AIRPORT DURING ASSEMBLY

FIG. 50.—COR-TEN STEEL SKELETON ON A DOUBLE DECKER BUS

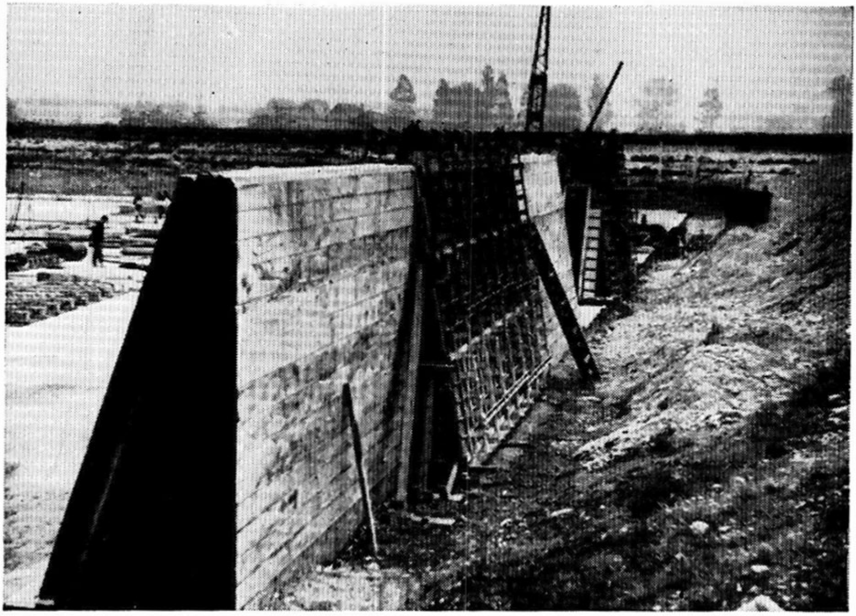


FIG. 51.—STEEL SHUTTERING: CHANNELS ARE 12 IN. × 12 IN.

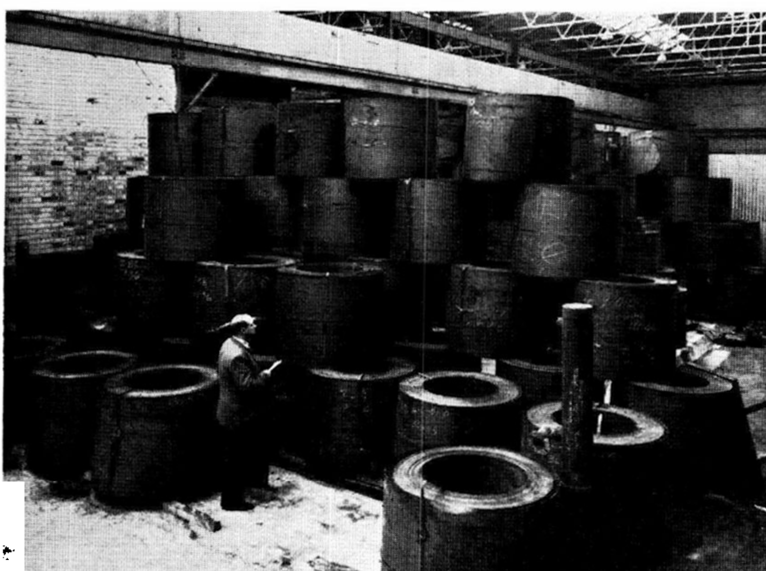


FIG. 52.—COILED SHEET MATERIAL

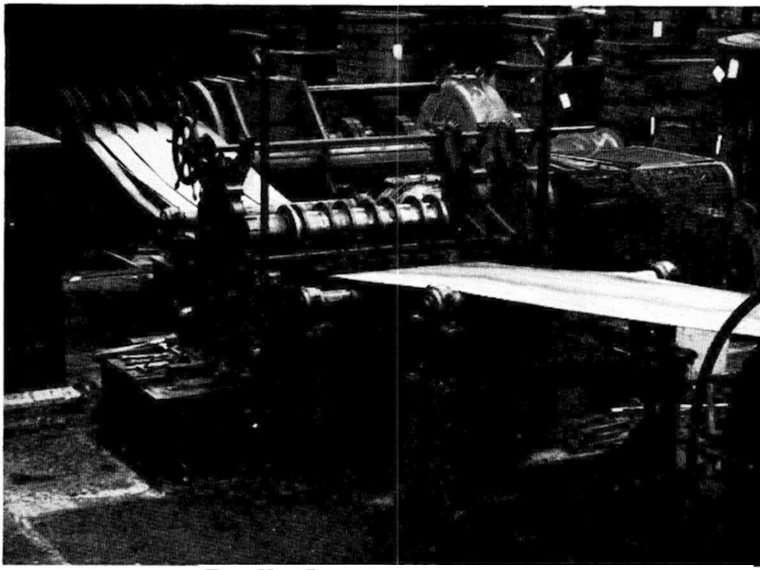


FIG. 53.—ROTARY SLITTING MACHINE

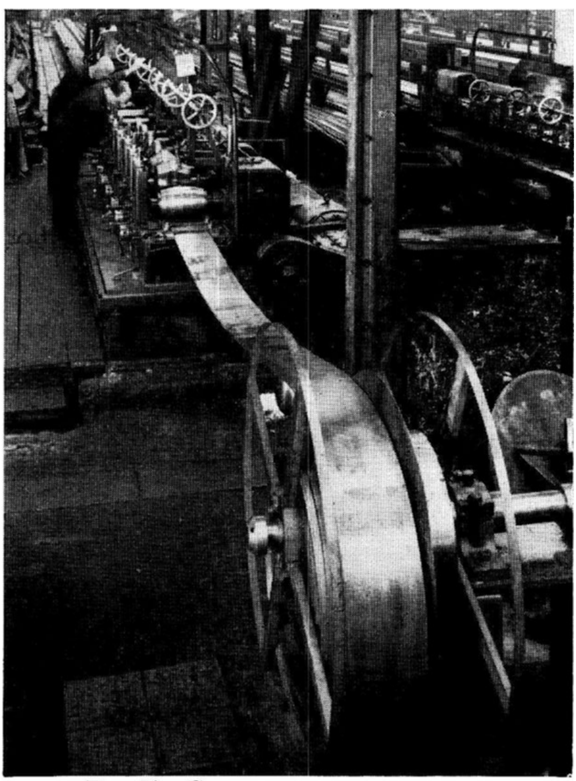


FIG. 54.—COLD-ROLLED FORMING MACHINE

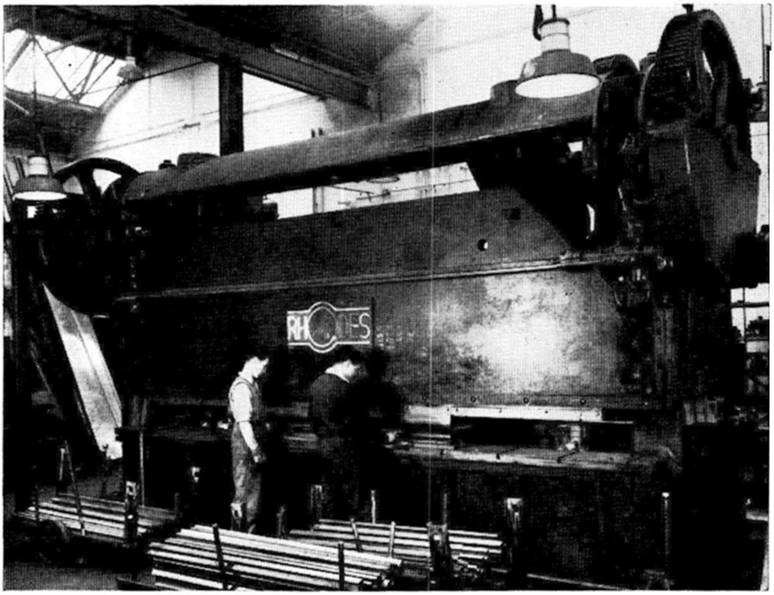


FIG. 55.—16-FT PRESS BRAKE

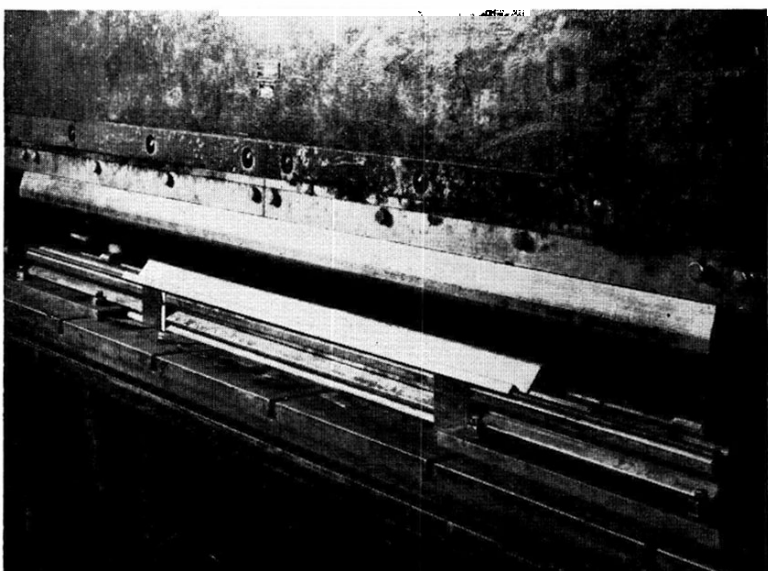


FIG. 56.—AIR BRAKE FORMING DIES

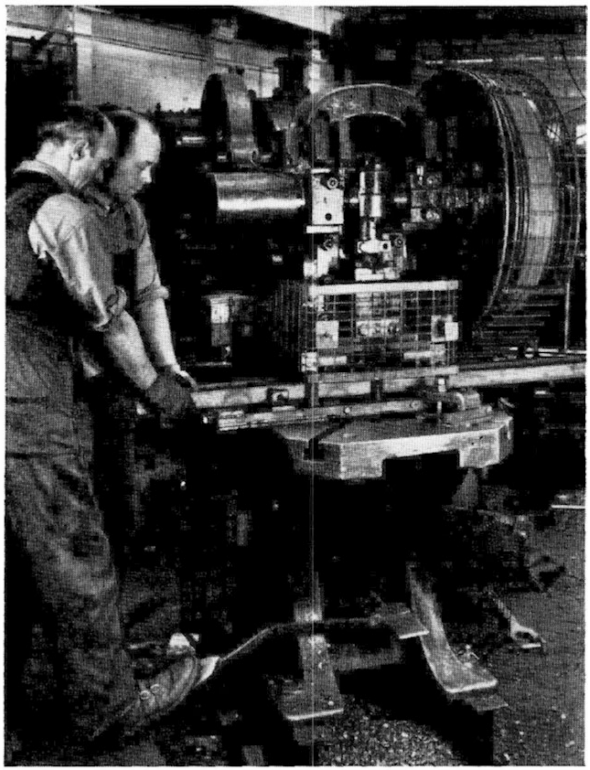


FIG. 57.—PIERCING PRESS

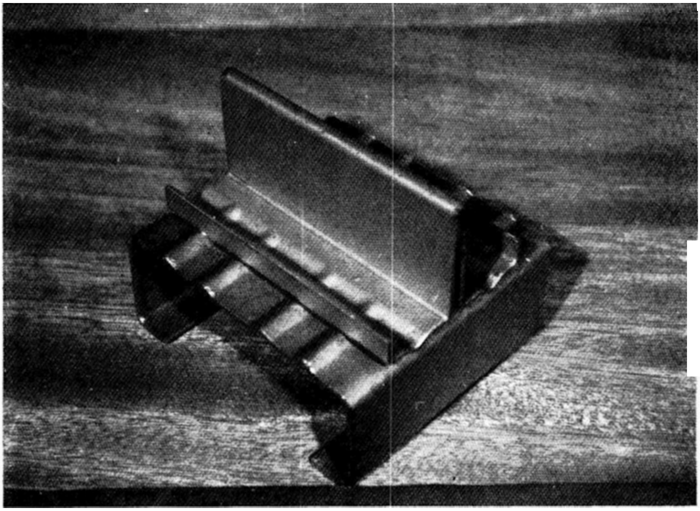


FIG. 58.—RIDGE WELDING OF COLD-ROLLED SECTIONS

FIG. 59.—RE-ENTRANT SECTION IN BUCKLED FORM AFTER LOADING TO DESTRUCTION

FIG. 60.—SAME SECTION AS IN FIG. 59, WITH CONCRETE TOPPING AFTER TESTING

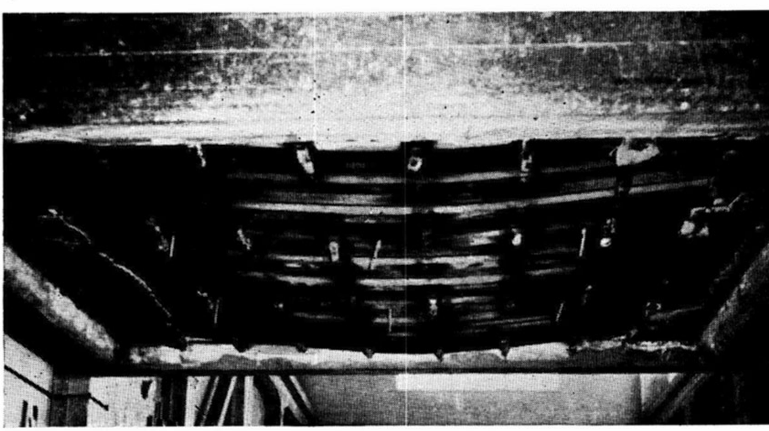


FIG. 61.—RESULTS OF A FIRE TEST ON SHUTTERING SECTION

It was designed on a gross section to a reduced working stress based on an adjusted empirical formula for the strength of corrugated iron.

88. The two points which lay below the Winter curve were for sections where the webs slope at 45° to the vertical. When calculating the effective width of the compression flange, it was usual to ignore buckling in the webs. However, when the slope of the webs was appreciable, buckling of the webs and reduction in the stiffening of the edges of the flange occurred and this led to an apparent reduction in the effective width of the compression flange as calculated by the simple theory. It was suggested that

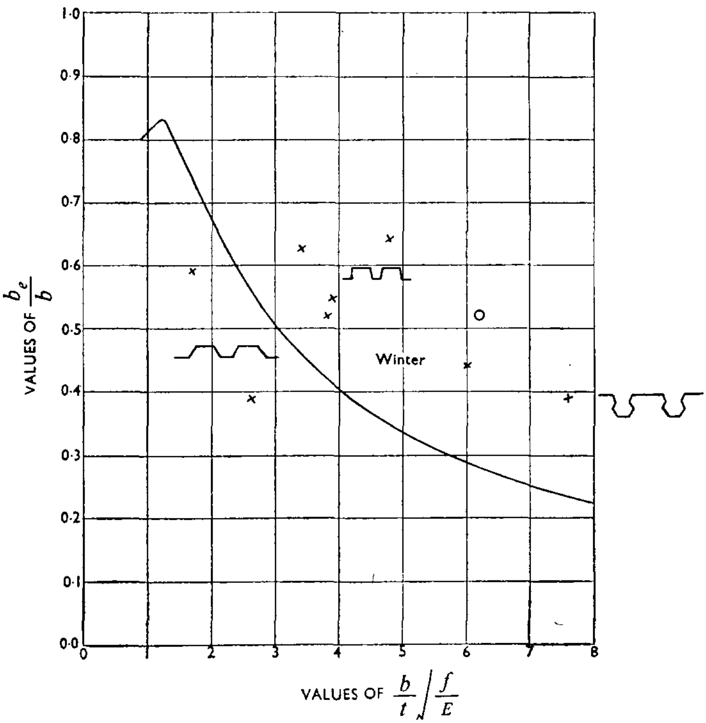


FIG. 41.—EXPERIMENTAL TEST RESULTS FOR EFFECTIVE WIDTHS OF COMPRESSION ELEMENTS AT COLLAPSE, COMPARED WITH THE WINTER CURVE

the points lay below the line due to over-simplification of the calculations. Would the Author offer any comments on this point and also any advice on how to include the effect of sloping webs?

89. When designing roof and/or side wall cladding, it must be remembered that economy was related to the area covered as well as to the load carried per unit weight of steel. In other words, the sheet was not just a load-carrying element, but also a weatherproof membrane or covering. It was, therefore, necessary to compromise between structural strength and the desire to produce the maximum coverage. True economic design could only be achieved if inclined web members were used.

90. In Fig. 41 Mr Lewis had compared his test results with the Winter formula quoted by the Author. However, this curve had been superseded by a more convenient formula in the design manual of the American Iron and Steel Institute. That formula

was supported by Winter and had the advantage that it did not require a bridging curve in the higher range of effective widths. Fig. 42 showed the two Winter curves, the revised curve marked *A* and the old curve marked *B*, the curve based on the Author's Fig. 15 and also the effective width curve abstracted from the addendum to B.S.449 on cold-rolled sections.

91. The Chilver curve agreed very closely with Winter's revised curve. However, the British Standard curve was more conservative. Perhaps the Author would care to comment.

92. Prof. Chilver concluded that "the structural problems of thin wall steel members are due to the relative thinness of the material". This led logically to the consideration of composite action using the expensive sheet in conjunction with cheaper and bulkier

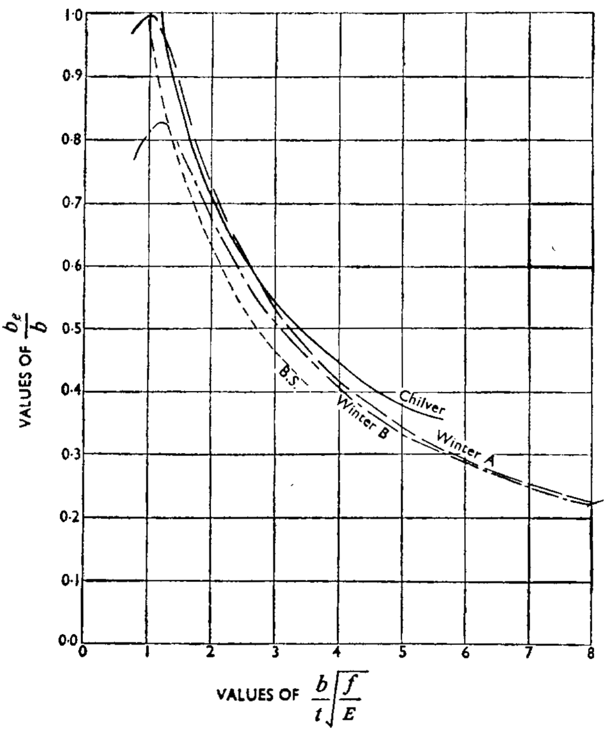


FIG. 42.—COMPARISON OF VARIOUS EFFECTIVE WIDTH CURVES

material which, besides cheapening the overall product, could combine the advantages of thermal and sound insulation, load spreading, etc. Could Prof. Chilver give any figures as to the bond strength which was required between thin wall sections and the bulk filling materials to suppress local buckling which was, in effect, the root of the problem? In this context, Mr Lewis would draw attention to the re-entrance section shown in Fig. 41. This section was to be filled with concrete and was made with re-entrant corners to ensure that the steel and concrete did not separate under load. The designers were hampered by the lack of a full theoretical treatment.

Mr F. W. L. Heathcote (Technical Manager, Brockhouse Steel Structures, Ltd) said that his particular interest in the subject was that of practical design, but there were three points in the Paper itself on which he wished to comment. To emphasize the point of practical design, however, he showed a few slides dealing with a method of assembly which they had used for some time. This was ridge welding, done on an electrical resistance welding machine. The sections had ridges in them, which could be formed at the time of rolling without extra expense. In each of the two ridges formed on the T section shown in Fig. 58 were four portions which had been flattened out by this welding. Those eight points joined the two sections together and they were welded simultaneously in a machine cycle time of 3 sec. This emphasized a point which had been implicit in the remarks of other speakers, that the jointing and fabrication generally of cold-rolled sections was to some extent a specialized job. To make the most of the cold-rolled section it was necessary to make use of the smooth, clean surface provided. Resistance welding was a quick process which made use of this clean surface which was also an advantage, as Mr Creasy had pointed out, when applying protective treatment. Electric resistance welding of this type did not leave any residue to be removed before the protective treatment; but—and this was an important factor—it required expensive equipment and, as had been indicated by some of Mr Griffin's slides, the complete assembly required jigging, which again implied that large quantities were required in order to make the job competitive.

94. Fig. 43 showed a rather more elaborate form of ridge welding. Half of the column section, was resistance welded to the channel, and 12 points were joined at one

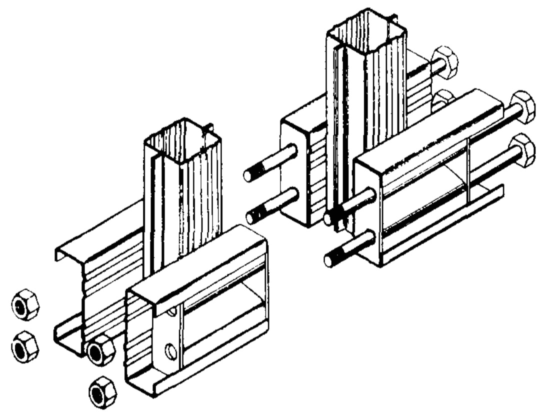


FIG. 43.—EXAMPLE OF RIDGE WELDING IN A LOW PITCH TRUSS

cycle of the machine. The same thing happened on the other side, and the two half-assemblies were then joined together by welding the short flanges.

95. Fig. 44 illustrated another form of resistance welding which could be used to join a channel to a column by arc welding a lug to the column and projection welding a flanged stud to the web of the channel. When bolted up the whole assembly was flush, which was often important when cladding prefabricated structures.

96. Turning to the points to which he wished to refer in connexion with the Paper, the first concerned equation 21, which had already been referred to. $\frac{L}{r_y} \times \frac{t}{b}$ was the fundamental parameter. He had been very interested in the chart showing that the ratio between the two terms, slenderness and thinness, was approximately 5. That led back to Mr Shearer Smith's specification, which had been read to the Structural Engineers many years ago. Mr Heathcote thought that this relation between slenderness and

thinness was a very important matter to practical designers, and he would like to suggest to the Author that the study of it should be carried much further.

97. In Mr Shearer Smith's stress table there was the slenderness ratio on the left and the next column gave the thinness ratio of the flange and the next the thinness ratio of the web. The ratio between the thinness of the flange and the thinness of the web was 3 and between the thinness of the flange and the slenderness ratio it was 5. It seemed that that old table was being vindicated by later research and Mr Heathcote was very pleased to see it.

98. The next point was the statement in § 29 that the mode of failure of a plate was independent of its length, or words to that effect. That led to a question about which he had been very anxious for many years, namely, whether or not they were justified in taking an effective length in dealing with columns subject to local buckling. He had always felt that it was not really justifiable to take the 0·85 or the 0·7 unless they were fairly sure that they were going to have a Eulerian type of failure; if they had a buckling type of failure it seemed to him that they should deduct only the half-waveform. Perhaps the Author would like to comment on that in his written reply.

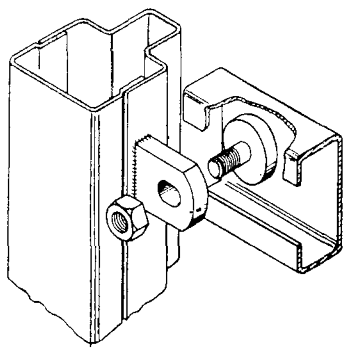


FIG. 44.—FLUSH CONNEXION USING A PROJECTION WELDED STUD

99. In § 17 it was stated that torsion led to longitudinal stress, and Sir Alfred Pugsley had asked whether or not this was a practical point. From Mr Heathcote's point of view it was a very important point in practice. If torsion led to longitudinal stress presumably the converse was true and longitudinal stress tended to produce twist. If they did not get such excellent diagrams of hardness as Mr Griffin had presented, if the tooling went wrong, if the hardness of the strip went wrong, or if it were tapering sideways, there would be different work-hardening on one side of the section from that on the other leading to a difference of longitudinal stress and therefore a certain amount of twist. It was, as Sir Alfred has said, easy to take out the twist, because it was of relatively small magnitude; but that raised the question of jigging. If twisted sections were put together without adequate jigging, the resulting welded assembly could be twisted. Practical users and practical fabricators should be very careful about this point, and it was most illuminating that the Author had brought it out in the Paper. It was a good thing that a theoretical basis for this well-known practical point had been demonstrated.

100. His final point was the difference between the American and the English modes of evaluating the reduction in the properties of an unstiffened section. The Americans said, when they produced their specification in 1946 after seven years' work at Cornell, that there was a fundamental difference between the performance of unstiffened flanges, as in the plain channel, and the stiffened flanges in the lipped or box channel. They

had said at the time that it was very important to appreciate this difference because the plain channel would fail at predictable stresses below the yield-point and they did not feel that the concept of an effective width was realistic. They felt that the stress or the modulus should be factored. They said that the resistance was equal to the stress multiplied by the modulus and that it did not matter whether one factored the stress or the modulus; one simply wanted to know what the reduction was.

101. In the case of the stiffened channel hardening occurred both at the lipped corner and at the corner of the web. The Americans said that there would be an ineffective portion of the flat areas due to buckling, and that buckling spread gradually according to the increase in stress until the yield-point was reached on the corners. They said that this was their basis for the evaluation of strength of stiffened sections. Mr Heathcote suggested to the Author that it was desirable to look at this from the point of view of whether there was really any important practical difference between those two viewpoints and whether there was any need to differentiate between the low stresses on the unstiffened flange and the ineffective areas on the stiffened flange.

Mr J. Horniblow (Research and Development Engineer, W. S. Atkins & Partners) took a little further the point made by Mr E. M. Lewis with reference to the possibility of composite action. Mr Lewis had shown on his curves a re-entrant section and Fig. 59 showed this in its buckled form after loading to destruction. The section shown had been designed for use as permanent shuttering and would normally have a concrete topping. The Fig. 60 showed the same section with its concrete topping after testing. In this case all the steel failed under a tension load, the compressive failure occurring in the comparatively cheap concrete. Sir Alfred Pugsley had referred to fatigue; they had done fatigue tests on this composite section which had shown after 50,000 cycles a reduction in strength of about 30%. One point which he had not heard mentioned that evening in relation to cold-formed sections of light gauge material was fire testing. The Fig. 61 showed the results of a fire test on the same permanent shuttering section where some reinforcing bars were incorporated in the concrete topping. This floor had withstood the standard one-hour fire test of the Fire Research Station. Perhaps the Author would comment on the fire hazard related to the use of cold-formed sections.

Mr D. S. Bondale (Structures Research Student, Imperial College, London) asked whether the Author could say something about the unavoidable effect of residual stresses resulting from the cold-forming operations. Research had been done in the United States on this subject, which showed that residual stresses could reduce the normal moment and/or thrust carrying capacity of hot-rolled sections by as much as 10–15%. Would this effect of residual stresses be of the same magnitude in the case of the cold-formed sections?

104. With regard to the stress/strain curves in Fig. 1 of the Paper, was the strain-hardening effect entirely absent? If not entirely absent at what strain would it appear?

105. Lastly, if cold-forming sections with a large number of corners resulted almost always in an increase in the tensile strength of the material over an appreciable area of the cross-section, could this possibly be taken into account in the design specifications at a later date, when the extent of this increase (in the tensile strength) was better known?

106. Purely academic as these queries may appear to be, yet they seem to need a closer look into, for evaluating their possible effect on the design rules presented by the Author.

Mr E. Ingerslev (Consulting Engineer) said he had been very pleased to note the close co-operation which apparently existed between the theoretical side and the manufacturing side, because that seemed to be essential in this field. He noticed the great variety of hardness along the shape of one of the profiles, and that would seem to point to the importance of testing, because any theoretical work on the strength of a particular profile must to a certain extent assume that it was a homogeneous material, which it was far from being in actual fact. He appealed to the industry to combine with the theoretical side to produce as soon as possible tables of tested cross-sections which could be used by the designer, because in this field the theoretical work might be far from the actual facts.

108. An earlier speaker had mentioned the difference between the simple channel and the one with the flange edges bent. He suggested that the fact was that what carried in all these profiles were the corners, where one could get near to the ultimate stress, the yield-point of the material. With the simple channel there were only two points to carry and as soon as the flanges gave way the whole section was affected, whereas with the corners of the flanges bent there were four points and one had something which would resist until a much later stage.

Dr M. S. G. Cullimore (Senior Lecturer in Civil Engineering, University of Bristol) wished to make one small point which he hoped was not too wide off the mark. Reference was made in § 47 of the Paper to structural connexions and to the use of high-strength bolts. It seemed to him that friction joints, where bolted joints were used in these structures, offered a very considerable advantage over ordinary fitted bolts. Where the load was transmitted by contact the bearing difficulties in thin walls were overcome. It also seemed possible that as the bolts were large in diameter compared with the thickness of the plate there would be a large area of contact and a reasonable amount of stress could be taken out before the hole was reached, so that in such cases it might possibly not be necessary to deduct for the full width of the hole. Also, the use of friction joints should reduce the connexion slip in continuous structures. Winter[34] in America had published some information on this topic, and found that there was no connexion slip at the design load provided the factor of safety was not less than 1·32 for painted or oiled surfaces and 1·54 for galvanized surfaces. He obtained as lower limits of the coefficient of friction 0·14–galvanized and 0·16–painted.

110. Addendum No. 1 to B.S.449 referred to the permissibility of using high-strength bolts and referred to B.S.3249, which dealt with the employment of these bolts in building construction but did not seem to concern itself at all with plates of the thickness here in question. The designer was faced with a lack of information if he wished to use high-strength bolts for thin-walled sections both in regard to design and in regard to the slip coefficient. Had anything been done on this aspect of the matter?

The following contributions were received in writing.

Dr A. R. Flint (Reader in Structural Steelwork, Imperial College) wrote that Professor Chilver was to be congratulated on his study of the difficult problems associated with thin steel elements—a study persistently maintained for ten years which had come to fruition in a form suitable for direct application to design. It was seldom found that such complex theoretical material was translated into simple design rules or tables and the Author's ingenuity must have been fully extended to achieve such neat results.

112. There were certain aspects of the treatment of stability presented which he should have liked clarified, particularly regarding the behaviour of beams. In Fig. 1 it was seen that a non-linear tensile stress-strain relation developed in typical thin-walled sections at a comparatively early stage. The lateral instability of beams was assumed to be an elastic problem, however, and Young's modulus had been taken to be constant. The test results shown in Fig. 20 showed, as might be expected, that collapse occurred at lower stresses than predicted from idealized elastic theory.

113. The departure from the theory might be due to the reduction of the tangent modulus in the tension flange (it would be seen that the divergence occurred at a stress of about 8 tons/sq. in. in both diagrams) combined with the influence of initial imperfections, which had been ignored. Had the Author attempted to allow for these effects, and had the simple method adopted for dealing with imperfections in B.S.153 been tried?

114. A further possible influence on buckling could be residual stresses and it would

be of interest to know whether controlled measurement had been made in these thin sections.

115. The Author had dealt separately with local and lateral buckling but did not discuss interaction effects. It was recalled that a study of this problem was published in another place some years ago following research at Bristol University. Was this interaction worth consideration in design and, if so, how should it be allowed for?

116. The lateral restraints due to the loading mechanism used in the beam tests would increase the collapse loads. It was not clear, however, whether the approximate theory plotted in Fig. 20 allowed for these. If not it would have been interesting to have the Author's views on the high failing loads in the high slenderness range.

117. Finally, the graph of steelwork weights shown in Fig. 23 showed the marked weight advantages to be gained by the use of thin materials. Unfortunately these advantages were less evident when costs were compared and it would be interesting to see a similar comparison plotted on a cost basis.

W. G. Beynon (Department of Materials Research, Admiralty) wrote that the base material used for the manufacture of cold-formed steel sections was described in the Paper as hot-rolled steel sheet. B.S.1449 contained a quality described as "24-Ton Steel Plate Sheet and Strip" and stated "This steel was intended for cold-rolled (formed) sections for structural purposes". There was no indication as to the condition in which the material was to be supplied to the producer of cold-formed steel sections i.e. whether the material was to be supplied, hot finished, pickled and oiled, cold-rolled and annealed, or cold finished, to give some of the possible conditions.

119. Some of the illustrations gave the impression of the material certainly being descaled and cold finished. He had learned since the lecture that material was supplied in any condition as required by the purchaser. This was presumably to suit fabrication methods and to have the necessary ductility to conform to the shapes being produced.

120. There were certain metallurgical aspects which needed consideration in view of the material being put into service in a cold worked condition, arising either from a cold rolling operation at the steelworks or from the bending operations in fabrication.

121. Such cold work might give rise to strain age embrittlement and possible fracture of a component under stress if the steel was not of stabilized quality. In addition the welding of unstabilized cold-worked material might give rise to soft spots in the vicinity of welds and to a strain aged zone at some distance from a weld.

122. These possibilities referred to above might be avoided by close collaboration between steelmaker, fabricator, and user whereby the appropriate steel quality and finish might be specified and the material treated by the user in the appropriate manner.

123. Such close collaboration may not always be possible and it is therefore suggested that the inclusion of a requirement for non strain ageing quality in B.S.1449 for material for cold-formed section would give an insurance against failure through strain age embrittlement in service.

Mr Peter Y. S. Pun (Gammon & Christiani-Nielsen, North Klang Straits Contract, Port Swettenham, Malaya) wrote that after reading Professor Chilver's interesting paper on the main structural design problems of thin-wall open-section members he had some comments on the general theory.

125. Professor Chilver reviewed the basic problems too briefly and some readers might have had difficulty in following them. For instance in § 18 "Tensile Members", no reference was given for readers to find the derivation of equation (9). Also he felt sure that Z section was not the only type to be used as thin-walled tensile members.

126. Between 1936–1940, Vlasov had established an engineering theory of open-section cylindrical thin-walled members—strength, stability and vibration[35, 36]. Umansky devised an engineering theory of closed-section thin-walled members in 1939[37] and revised that theory in 1940[38]. In 1956, Jay solved the dynamical stability problem[39].

127. By assuming that the projection of the shape of cross-sections remained unchanged after twisting (to replace the assumption in the beam theory that plane remained plane after bending) and assuming no shear strain at the middle surface of the cross-section, together with the general assumptions in the theory of thin shells, Vlasov had obtained the following equations:

$$EA\frac{d^2\zeta}{dz^2}+q_z = 0$$
$$EI_y\frac{d^4\xi}{dz^4}-q_x = 0$$
$$EI_x\frac{d^4\eta}{dz^4}-q_y = 0$$
$$EI_\alpha\frac{d^4\phi}{dz^4}-GI_z\frac{d^2\phi}{dz^2}+m = 0 \quad \ldots \ldots \ldots \ldots \quad (1)$$

where E = Young's modulus.
A = Area of cross section.
I_x, I_y = moment of inertia about x, y axes respectively.
G = shearing modulus.
I_z = moment of inertia about z axis.
q_z, q_x, q_y = linearly distributed loads in z, x and y directions respectively.
m = linearly distributed twisting moment about the shear centre axis.
ζ, ξ, and η = displacements of shear centre axis in z, x and y directions respectively.
ϕ = angle of twist.
I_α = a new geometric property. Vlasov had called "Sectorial Moment of Inertia" having L^6 dimension.

The value was defined by

$$I\alpha = \int_A \alpha_c^2 dA \quad \ldots \ldots \ldots \ldots \quad (2)$$

where α_c was defined by equation (2) in Professor Chilver's paper.

128. By using the following four generalized forces,

$$N = \int_A \sigma 1 dA = EA\frac{d\zeta}{dz} = EA\zeta'$$
$$M_x = \int_A \sigma y dA = -EI_x\frac{d^2\eta}{dz^2} = -EI_x\eta''$$
$$M_y = -\int_A \sigma x dA = -EI_y\frac{d^2\xi}{dz^2} = -EI_y\xi''$$
$$B = \int_A \sigma\alpha dA = -EI_\alpha\frac{d^2\phi}{dz^2} = -EI_\alpha\phi'' \quad \ldots \ldots \quad (3)$$

where

$$A = \int_A 1^2 dA,\ I_x = \int_A y^2 dA,\ I_y = \int_A x^2 dA,\ I_\alpha = \int_A \alpha^2 dA$$

and B a new generalized force which Vlasov had called "Bi-moment" (having lb$-$in.^{2} dimensions) due to the twist of the cross section and which was in a state of static equilibrium, one could obtain

$$\sigma = \frac{N}{A}-\frac{M_y}{I_y}x+\frac{M_x}{I_x}y+\frac{B}{I_\alpha}\alpha \quad \ldots \ldots \ldots \ldots \quad (4)$$

From equation (4), together with equilibrium equation of a shell element

$$\frac{\partial(\sigma t)}{\partial z}+\frac{\partial(\tau t)}{\partial s} = 0$$

one obtained the shear stress

$$\tau = -\frac{Q_X S_Y}{I_Y t} - \frac{Q_Y S_X}{I_X t} - \frac{K S_\alpha}{I_\alpha t} \quad \ldots \ldots \quad (5)$$

where t was the thickness of the member

$$\left.\begin{aligned} S_X &= \int_{S_A}^{S} y dA = \int_{S}^{S_B} y dA \\ S_Y &= \int_{S_A}^{S} x dA = \int_{S}^{S_B} x dA \\ S_\alpha &= \int_{S_A}^{S} \alpha dA = \int_{S}^{S_B} \alpha dA \end{aligned}\right\} \quad \ldots \ldots \quad (6)$$

$$\left.\begin{aligned} Q_X &= \int_A \tau t dx = -EI_Y \xi''' \\ Q_Y &= \int_A \tau t dy = -EI_X \eta''' \\ K &= \int_A \tau t d\alpha = -EI_\alpha \phi''' \end{aligned}\right\} \quad \ldots \ldots \quad (7)$$

In equation (6), s_A and s_B were the extreme points of the cross section.

129. From equation (5) and (7), one obtained

$$\begin{aligned} Q_X &= -M'_Y \\ Q_Y &= M'_X \\ K &= B' \end{aligned}$$

130. From the above, it could be seen that in solving basic design problems of thin-walled members, besides finding the forces and geometric properties in the plane theory of strength of materials, one also had to find the new forces—B and K, and new geometric properties—α, I_α, S_α.

131. The bi-moment B could be obtained by solving the last part of equation (1).

$$EI_\alpha \phi^{IV} - GI_z \phi'' + m = 0 \quad \ldots \ldots \quad (9)$$

which could be transformed in terms of B

$$B'' - \frac{GI_z}{EI_\alpha} B = m$$

by introducing

$$\lambda = \sqrt{\frac{GI_z}{EI_\alpha}}$$

one could write

$$B'' - \lambda^2 B = m \quad \ldots \ldots \quad (10)$$

132. The general solution of equation (10) was

$$B = C_1 . \sinh \lambda z + C_2 . \cosh \lambda z + B_0 \quad \ldots \ldots \quad (11)$$

where C_1 and C_2 were constants which could be determined by the loading conditions and B_0 was the particular solution. If m varied linearly, $B_0 = \frac{m}{\lambda^2}$.

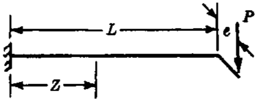
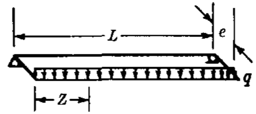
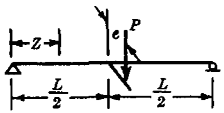
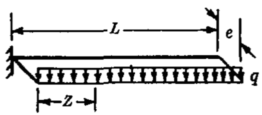
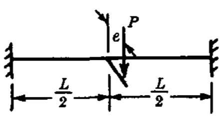
133. The general solution of equation (9) was

$$\phi = D_1 + D_2 z + D_3 . \sinh \lambda z + D_4 . \cosh \lambda z + \phi_A \quad \ldots \ldots \quad (12)$$

where D_1, D_2, D_3 and D_4 were constants which could be determined by the boundary conditions and ϕ_A was the particular solution depending on the loading condition.

134. Some of the solutions of B and K were stated in Table 1.

TABLE 1

	Bi-moment B	K
	$-\dfrac{Pe\sinh\lambda(L-Z)}{\lambda\cosh\lambda L}$ when $Z=0$, $B=B_{\max}$	$\dfrac{Pe\cosh\lambda(L-Z)}{\cosh\lambda L}$ when $Z=0$, $K=K_{\max}$
	$-\dfrac{qe}{\lambda^2\cosh\lambda L}[\lambda L\sinh\lambda(L-Z)-\cosh\lambda L+\cosh\lambda Z]$ when $Z=0$, $B=B_{\max}$	$\dfrac{qe[\lambda L\cosh\lambda(L-Z)-\sinh\lambda Z]}{\lambda\cosh\lambda L}$ when $Z=0$, $K=K_{\max}$
	$\dfrac{Pe\sinh\lambda Z}{2\lambda\cosh\frac{\lambda L}{2}}$ when $Z=\frac{L}{2}$, $B=B_{\max}$	$\dfrac{Pe\cosh\lambda Z}{2\cosh\frac{\lambda L}{2}}$ when $Z=\frac{L}{2}$, $K=K_{\max}$
	$\dfrac{qe}{\lambda^2}\left[1-\dfrac{\cosh\lambda\left(\frac{L}{2}-Z\right)}{\cosh\frac{\lambda L}{2}}\right]$ when $Z=\frac{L}{2}$, $B=B_{\max}$	$\dfrac{qe}{\lambda}\cdot\dfrac{\sinh\lambda\left(\frac{L}{2}-Z\right)}{\cosh\frac{\lambda L}{2}}$ when $Z=0$, $K=K_{\max}$
	$\dfrac{Pe\left[\cosh\lambda Z-\cosh\lambda\left(\frac{L}{2}-Z\right)\right]}{2\lambda\sinh\frac{\lambda L}{2}}$ when $Z=0$ and $Z=\frac{L}{2}$, $B=B_{\max}$	$\dfrac{Pe\left[\sinh\lambda Z+\sinh\lambda\left(\frac{L}{2}-Z\right)\right]}{2\sinh\frac{\lambda L}{4}}$ when $Z=0$ and $Z=\frac{L}{2}$, $K=K_{\max}$

135. In the case of a longitudinal force acting at the end section, B_0 at that section could be obtained from

$$B_0 = P\alpha_p \qquad (13)$$

where α_p was the "sectorial area" at the point of application. If P was acting at a point where α was zero, no torsion would be produced. For example, when P was acting at O in Fig. 45 where the "sectorial area" α was zero, the Z section member was subject to

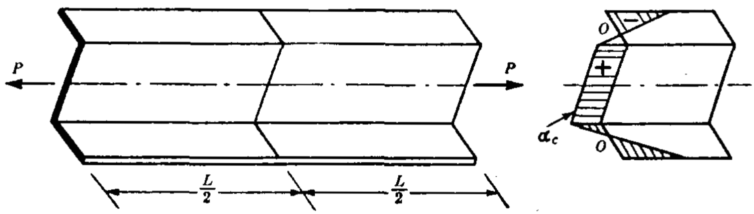


FIG. 45.—TWISTING CHARACTERISTICS OF A Z-SECTION DUE TO LONGITUDINAL FORCE

bending and pull but not torsion, but when P acted at the centroid of the Z section where $\alpha \neq 0$ then torsion would be produced.

136. Regarding the stability problems, Vlasov had obtained the following equations for a thin-walled rod subject to a longitudinal force P and end bending moments M_x and M_y (see Fig. 46).

$$\left.\begin{aligned} &EI_y\xi^{IV}+P\xi''+(M_x+y_0P)\phi'' = q_x \\ &EI_x\eta^{IV}+P\eta''+(M_y-x_0P)\phi'' = q_y \\ &(M_x+y_0P)\xi''+(M_y-x_0P)\eta''+EI_\alpha\phi^{IV}+(r_0{}^2P+2\beta_xM_y-2\beta_yM_x+GI_z)\phi'' = m \end{aligned}\right\} \quad . \; (14)$$

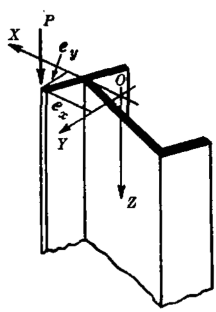


FIG. 46.—THE CRITICAL ECCENTRIC LOAD OF A THIN-WALLED ROD

where $r_0{}^2$, β_x and β_y were defined by

$$\left.\begin{aligned} r_0{}^2 &= \frac{I_x+I_y}{A}+x_0{}^2+y_0{}^2 \\ \beta_x &= k_x-x_0 \\ \beta_y &= k_y-y_0 \end{aligned}\right\} \qquad (15)$$

and $x_0\,y_0$ could be obtained from equations (1) in the Paper, k_x and k_y were the co-ordinates of the centre of "extended stability circle"

$$\left.\begin{aligned} k_x &= \frac{1}{2I_y}\int_A x(x^2+y^2)dA \\ k_y &= \frac{1}{2I_x}\int_A y(x^2+y^2)dA \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (16)$$

when $q_x = q_y = m = 0$,
equation (14), together with the boundary conditions would define the shape of the rod after losing stability due to combined bending and torsion for any type of cross section. If the rod was hinged at both ends, one could obtain the following determinant equation

$$\begin{vmatrix} P_y - P; & 0; & -(Py_0 + M_x) \\ 0; & P_x - P; & Px_0 - M_y \\ -(Py_0 + M_x); & Px_0 - M_y; & P_\alpha r_0^2 - (Pr_0^2 + 2\beta_x M_y - 2\beta_y M_x) \end{vmatrix} = 0 \quad . \quad (17)$$

where P_x, P_y were the Euler critical loads; P_α was the critical load causing torsional instability.

$$\left.\begin{aligned} P_x &= \frac{EI_x\pi^2}{L^2} \\ P_y &= \frac{EI_y\pi^2}{L^2} \\ P_\alpha &= \frac{1}{r_0^2}\left(\frac{EI_\alpha\pi^2}{L^2} + GI_z\right) \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \quad (18)$$

137. In equation (17), taking $M_y = -Pe_y$, $M_x = Pe_y$ (see Fig. 46) one could obtain the equation to determine the critical compression or tensile * load P

$$\begin{vmatrix} P_y - P; & 0; & -P(y_0 - e_y) \\ 0; & P_x - P; & P(x_0 - e_x) \\ -P(y_0 - e_y); & P(x_0 - e_x); & P_\alpha r_0^2 - P(r_0^2 + 2\beta_x e_x + 2\beta_y e_y) \end{vmatrix} = 0 \quad . \quad (19)$$

138. Expanding the above equation one obtained

$$\begin{aligned} (P_x - P)(P_y - P)[r_0^2(P_\alpha - P) - 2(\beta_x e_x + \beta_y e_y)P] \\ -[(y_0 - e_y)^2(P_x - P) + (x_0 - e_x)^2(P_y - P)]P^2 = 0 \end{aligned} \quad . \; . \; . \; . \; . \quad (20)$$

139. From equation (20), the least one of the three roots was the critical load acting at a point E (e_x, e_x).

140. The problems of vibration, dynamical stability and closed section thin-walled rods were beyond the scope of the Paper and would not be discussed here. Also it was impossible to discuss the application of the above theory in solving some important structural members such as braced or battened members, continuous members etc. here due to limitations of space.

141. Finally, he would like to know whether the author observed the validity of St Venant's principle in the theory of thin-wall members.

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* According to Euler's theory, tensile force would not cause instability of the rods but according to Vlasov's theory, if the tensile force was acting at a point beyond the "Extended Stability Circle" it might cause instability of the rods.
The equation for the "Extended Stability Circle" was

$$(e_x - x_0 - \beta_x)^2 + (e_y - y_0 - \beta_y)^2 = r_0^2 + x_0^2 + y_0^2 + 2(x_0\beta_x + y_0\beta_y)$$

The Author, in reply, wished to thank all those who had contributed to the discussion of the Paper; some of his colleagues in the cold-formed steel industry had rightly pointed out other structural problems of importance, and he was most grateful to them for the pains taken in presenting their comments. It had always been a great source of interest to the Author to work in this field; it had always been impressed on him that simple solutions were required to practical design problems, and this he had found a stimulus to research.

143. The Author was particularly grateful to Sir Alfred Pugsley for his introductory remarks in the discussion. Sir Alfred had commented on the interest created by new structural materials. The study of metal structures had its origins in the cast-iron frames of the early 19th century, and later in wrought-iron frames. Further progress in structural mechanics was made with the introduction of mild-steel at the turn of the century. In the early 1900's, aluminium slowly replaced timber and steel as a material for aircraft structures, and many of the most important studies of structural stability arose out of aluminium structures. The use of thin steel structures over the past 20 years or so introduced new problems; the structural mechanics of thin-walled sections was a relatively new field of knowledge, although such problems were studied in Russia between the two world wars.

144. The practical importance of twisting in tensile members had been questioned by Sir Alfred Pugsley. The twisting of a Z-section in axial tension was discussed in § 18, and illustrated in Fig. 6; this example was introduced by the Author to show that axial tension was not a simple loading condition for a thin-walled section. The practical importance of this example was that considerable thought must be given to connexion details by the designer; heavy connexions might help to suppress twistings, but considerable (and perhaps visible) twistings could still occur between connexions. If axial loads were applied at specific points—points of zero warping—of the cross-section, then no twisting of the section occurred; a knowledge of the basic structural mechanics of thin-walled sections was essential therefore to the designer. Mr Heathcote, in his contribution to the discussion, had suggested that certain states of residual stress might lead to initial twistings in members; such twistings were important from the fabrication point of view.

145. The Author was grateful to Sir Alfred Pugsley for expanding on the structural efficiency of thin-walled members. Fig. 22 was a comparison of cold-rolled and hot-rolled channel columns; this comparison was not necessarily typical of all cross-sectional forms. The cold-rolling technique lent itself very much to the forming of almost any cross-sectional profile; in general, more efficient cross-sectional forms could be made in cold-rolled sections, than in hot-rolled. A comparison of efficiencies was made even more difficult when higher strength materials were used.

146. Sir Alfred Pugsley asked about future research problems in thin steel sections. Most of the structural problems of cold-formed sections could be considered either as problems of

(i) material (such as mechanical properties, corrosion, etc.),
(ii) structural strength (such as local instability, torsion-flexure, etc.),
(iii) design (such as factors of safety, design methods, etc.).

147. So far as material problems were concerned, more attention would have to be given to the mechanical properties of materials most suited to the cold-forming process; such problems were particularly important in the case of high-strength materials. Mr Griffin's comments showed the important effect of the cold-rolling (as opposed to brake-pressing) operation on the hardness of sections; the importance of this effect on structural strength should be explored carefully. Such work-hardening might introduce other problems; Sir Alfred Pugsley very rightly warned of the possibility of shearing fatigue of the corners of cold-rolled sections. The effect of work-hardening might be to narrow the margin between initial yielding and ultimate failure of a tensile member —in some applications this might be undesirable.

148. In the field of structural strength of cold-formed steel sections, work in the past had been concentrated largely on the buckling of columns and beams, and on the strength of connexions. In this field, more research on

(i) the local buckling of wide component plates, such as those used in floor systems,
(ii) the torsional-flexural behaviour of beams under various loading conditions (including the strength and stiffness of purlins), and
(iii) the behaviour of full-size structures of the cold-formed steel type,

would add considerably to our present state of knowledge.

149. Considerable effort had been made in the past to apply research results to the formulation of design methods; a design specification for cold-formed steel structures had been prepared*, and a design handbook had been written. The design specification would require careful revision during the next few years, so that as the field of cold-formed steel structures developed, design methods were available to deal with the most relevant problems. Now that design methods were more fully defined, it should be possible to make comprehensive surveys of the efficient use of cold-formed steel sections —from both the standpoints of weight and cost; for this purpose, the development of computer programmes for the design of components and frames would be of great help.

150. Mr Creasy, who had considerable experience in the practical application of cold-formed steel sections, had made a number of important points. His information on the efficiency of thin steel trusses was particularly interesting. He noted too the complementary use of cold-formed and hot-rolled steel sections, notably purlins and floor joists of thin steel, used in conjunction with hot-rolled frames. Mr Creasy's additional comments on corrosion were particularly welcome; his point that the protective coating must itself be maintained is an important one. The Author was particularly grateful to Mr Creasy for his illustrations of the use of Addendum No. 1 to B.S.449; the examples quoted showed most of the important design concepts in cold-formed steel sections.

151. The Author was very grateful to Mr Shearer Smith for showing a number of practical applications for thin steel sections, from a whole structure to complementary construction. An important application of cold-formed steel sections was in the frames of transport vehicles.

152. Dr Kenedi's work on connexions was particularly interesting, and it was hoped that he would be able to pursue his studies in this direction. The analysis of connexion behaviour would appear to depend very much on the assumed mode of failure of the connexion; perhaps it would be possible for Dr Kenedi to enlarge on this problem elsewhere.

153. Mr Griffin had very kindly enlarged on the details of the cold-forming techniques, and the illustrations he had provided were most interesting. Mr Griffin's study of the hardness properties in various parts of the cross-section of a member was the first to be published—to the Author's knowledge. In § 81 he had made an important point that the work-hardening effect reduced the margin between yield and ultimate. The problem of work-hardening was undoubtedly one requiring much further study.

154. The Author was very grateful for Mr Lewis's comments on the Paper. As Mr Lewis had noted, the Author's work was limited to thinness ratios of about 75; it was true that large amounts of steel sheet were being used at thinness ratios of 250 or more. In many such problems, however, structural design was really rather difficult; in general the Author would agree that the wider sheets required closer study.

155. Mr Lewis noted that the Winter curve[1] represented rather conservatively results of tests on corrugated sheets; this would agree with the general notion that the Winter curve was anyway conservative. So far as the design of corrugated sheets was concerned it would not be strictly relevant to apply flat-sheet design formulae, and it

* *Loc. cit.*, p. 233.

might also be dangerous to extend the relatively simple ideas of single plates to such sheets; much research had been done in the corrugated sheet field.

156. Mr Lewis raised the question of coverage as opposed to structural weight efficiency. The problem of coverage could be taken account of in structural efficiency calculations. This was particularly so in beam problems, where the spans and spacing of the beams were important from the standpoint of efficiency.

157. A further point made by Mr Lewis was the conservativeness of the local buckling curve in the British Standard. The curves suggested by the Author and by Winter—these were shown in Fig. 42—were average curves through experimental results. The curve used in preparing the British Standard, on the other hand, was drawn specifically on the conservative side of the experimental results; this accounted for the relatively conservative B.S. curve.

158. The comments made by Mr Lewis on composite construction were particularly interesting. Mr Lewis posed the problem of local buckling with one side of the component plate embedded in concrete. To the Author's knowledge, this problem had not been studied comprehensively—although some studies of particular problems might have been made. The amount of bonding required to prevent local buckling was probably not very great, but the Author was unable to reply to this question quantitatively. Similar problems had been met in aircraft practice—especially in relation to sandwich construction, where the skin must be attached to the inner core of the sandwich.

159. The Author was very grateful to Mr Heathcote for his comments; Mr Heathcote had been as deeply concerned with the technical problems of cold-formed steel sections as perhaps any other structural engineer in this country, and it was very good to have comments from an engineer of his wide experience in this field. Mr Heathcote had emphasised the importance of jointing techniques, especially ridge welding; this showed the extent to which design details in cold-formed steel sections could differ from those in hot-rolled sections. The torsional-flexural buckling parameter was extremely important in the design of thin-walled columns of open section. The Author, together with Mr C. P. Hone (now with Ove Arup & Partners), had made a more detailed study of the torsional-flexural buckling of a wide range of columns, and it was hoped to publish this very shortly.

160. Mr Heathcote raised an important problem in the "effective" length of a thin-walled open-section column. If there were no interaction of local and column buckling, then "effective" column lengths could be treated in the conventional way—allowance being made for torsional-flexural buckling. In fact, local instability and column buckling probably interacted to some extent; this problem of interaction was intractable in most practical cases, so that very little was known of the interaction effect. At present, one relied very much on test data to define interaction effects. Mr Heathcote's comments on residual stresses, and their tendency to initially twist cold-rolled sections, were very interesting; residual stresses could also distort the cross-sectional form locally, although a general condition of twisting was possible.

161. Mr Heathcote had very correctly pointed to an important difference of design approach between the American and British design specifications, when he raised the question of stiffened and unstiffened elements. Considered in isolation there was clearly a strength advantage to be derived from providing a stiffening lip to an unstiffened element. The local buckling strength of a section consisting of a number of component plates was, however, basically a function of the interaction of the component plates. Researches on both steel and aluminium sections showed that the local buckling behaviour was basically the same, regardless of whether some components were stiffened or unstiffened. The discussion of the problem in terms of isolated plates was simply a design device; this simplified design approach gave results in reasonable agreement with more rigorous (and more complicated) methods. It was in fact uneconomic to use wide unstiffened elements either in columns or beams; although this difference between British and American practice might persist, it was probably not all that important.

162. Mr Horniblow had very kindly enlarged on some of the problems raised by Mr Lewis. It would be interesting to know more of the fatigue failure mentioned by Mr Horniblow—presumably fatigue occurred in the concrete topping in this case. Mr Horniblow's question on the fire hazards of cold-formed sections was a difficult one to answer; protective treatments required for cold-formed steel sections to provide fire resistance were in general similar to those required for other types of steel structures.

163. Mr Bondale had enquired of the effects of residual stresses and the possibly detrimental effect of the cold-working process. As Mr Griffin had pointed out, the tensile strength of a section could be increased appreciably by the cold-rolling process. The effect on comprehensive strength was not known at present; this problem would probably receive more attention in future. So far as strain-hardening on the stress/strain curve was concerned this occurred at the usual strains of mild-steel—this range of strains was not shown in Fig. 1. There was no reason why the cold-working effect should not be taken into account in the design procedure. The main difficulty lay in accurately predicting this effect.

164. Mr Ingerslev very rightly suggested that the cold-working effect should be thoroughly studied by experiment. Much of the B.S.449, Addendum No. 1, was based on direct testing, and this approach would continue to be used. As Mr Ingerslev had indicated, the advantage in stiffened plate elements was that two remote areas of the element could carry heavy stresses; this was particularly important in simple beams and columns; it meant that the unstiffened element was an inefficient load carrying component.

165. Dr Cullimore had raised the question of what type of bolted connexions was most suitable for cold-formed steel construction. His remarks were very welcome, and the industry should no doubt consider these very carefully. In § 47 the Author had referred to high-strength steel bolts; these comments were written in 1960 at the time of the preparation of the Paper; there were now indications that these views no longer reflected the ideas of the designers, and a more careful re-appraisal of this problem would be necessary.

166. The Author was very grateful to Dr Flint for his kind remarks. The Author's treatment of the lateral instability of fabricated I-beams was a very approximate one; as Dr Flint had correctly pointed out, the effects of premature tensile yielding, initial imperfections, and lateral restraints had not been taken account of. The lateral buckling of cold-formed steel sections presented so many problems that it was unprofitable to refine the analysis too highly. The more general problem of torsional-flexural behaviour of thin-walled unsymmetrical beams still remained largely unsolved; in Fig. 20 the relatively high stresses attainable for long beams were due to the lateral restraining effects of the loading system. No measurements were made of residual stresses; in fact, some of the tests on beams were of pressed (as opposed to rolled) sections—where the residual stresses would anyway be not too high. Dr Flint raised the question of the cost of cold-rolled steel work. This problem is rather outside the scope of the Paper. Costs had been studied to some extent by other workers, and some mention of these had been made already by Mr Creasy in his contribution to the discussion.

167. Mr Beynon raised important questions in relation to the material used in cold-formed steel practice; as he pointed out, the material was supplied to the purchaser in any required condition. By the very nature of cold-formed sections there was less standardization than in hot-rolled sections; this certainly applied to shapes, and might also be true of materials to some extent. Mr Beynon's remarks showed how important it was that the most appropriate steels should be used in the cold-forming process; much more thought should be given to this aspect, especially if use was to be made in design of the cold-working effects.

168. Mr Pun had enlarged very usefully on the theoretical aspects of the elastic behaviour of thin-walled open sections. The work of Vlasov, and others in the Russian school, was not widely known in the West, and it was very relevant to point this out. Table 1 of Mr Pun's contribution was useful in indicating the forms of the generalized

forces B and K. Mr Pun had asked whether the Author observed the validity of St Venant's principle; this problem had not arisen in dealing with the basic case of local buckling; as demonstrated by Vlasov, conventional ideas of the theory of solid beams required revision in the case of thin-walled open-section beams. St Venant's principle was valid only in a more localized sense. •

169. In conclusion the Author wished to thank all those who had taken part in the verbal and written discussion of the Paper. The discussion as a whole showed the wide range of structural problems in cold-formed steel sections; there were many considerations which faced the designer of cold-formed steel structures, and the Author hoped that the Paper, taken together with the discussion, had indicated the most relevant problems.

References

34. G. Winter, "Light gauge steel connexions with high-strength, high-torqued bolts". I.A.B.S.E. Publications, vol. 16, 1956.
35. V. Z. Vlasov, "Analysis of rib-stiffened shells and beams of thin-wall construction subject to bending and torsion". Prockt i standart, 10, 1936 (in Russian).
36. V. Z. Vlasov, "Thin-walled elastic rods, Gosslroiizdat, 1940 (in Russian).
37. A. A. Umansky, "Torsion and bending of air-frames" Oborongiz, 1939, (in Russian).
38. A. A. Umansky, "Direct stress due to the twist of aeroplane wings", Technika vozdushenogo flota, No. 12, 1940 (in Russian).
39. B. M. Jay, "Theory of dynamical stability of elastic thin-walled rods", Journal Physics, Sinica, vol. 12, 1956 (in Chinese).