Fire resistance of encased steel stanchions

by

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and

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Mr L. R. Creasy (Assistant Chief Civil Engineer, Ministry of Public Building and Works) said that the tests described in the Paper made an important contribution to knowledge of the fire resistance of encased stanchions and their load-carrying capacity in both the 'fired' and 'unfired' condition. The tests must first of all be viewed in terms of the present design requirements, as given in B.S.449, and appeared to indicate that some modifications were required.

69. In terms of the present rules, the Paper showed that there was no reason to insist on a minimum of 2 in. of cover. A cover of only 1 in. seemed to provide a satisfactory protection for two hours of fire resistance. This was also confirmed in the group of tests with 1-in. cover reinforced with a very lightweight mesh. Such a mesh would be easy to fix and relatively cheap as compared with the intricate arrangement of stirrups which was required at present under B.S.449.

70. Another important contribution from these tests related to the lightweight casings which were made with expanded clay or slag aggregates. These tests appeared to show that the carrying capacity of an encased column in both the 'fired' and 'unfired' condition was not adversely affected when lightweight aggregates were substituted for gravel.

71. Perhaps the most important contribution came from equation (1), the basic conception of which was used in the design of a reinforced-concrete column. This equation stated that the ultimate load-carrying capacity of a cased stanchion was calculated from the sum of the areas of concrete and steel multiplied by their respective yield stresses. In terms of fire resistance this gave a considerable flexibility in design to meet the requirements of aesthetics, fire resistance and carrying capacity not formerly possible by conventional rules.

72. But surely the most startling contribution was the application of equation (1) to the ultimate load of a cased stanchion in the 'unfired' condition. In this series of tests the yield stress of the steel and concrete were 15 tons/sq. in. and 3000 lb/sq. in. respectively 'before firing'. No reduction was required for the slenderness of the column since the L/D ratio was less than 20.

73. Taking the example of an 8 in. $\times$ 6 in. stanchion in a casing of 12 in. $\times$ 10 in. overall size and applying these yield values to the area of steel and total area of concrete, the ultimate carrying capacity of the cased strut was 300 tons approximately, equally divided between the steel and the concrete. This appeared to agree very closely with the test result in the 'unfired' condition. Similar agreement existed with the other examples and appeared to indicate that cased steel columns could be designed in the same way as for reinforced-concrete columns.

74. At the present time hollow board casings were more economical than concrete

casings because conventional rules did not allow an efficient contribution to the load from the concrete. These tests appeared to reverse this situation. Applying a load factor of 2 to the example indicated, the safe carrying capacity of the cased stanchion was 150 tons, which was approximately 50% in excess of that at present allowed under the Code rules.

75. In the form in which equation (1) was applied in reinforced-concrete design, the whole cross-sectional area of the concrete was included in the calculation without excluding the area of concrete cover as now proposed for cased steel design. An allowance for the full area appeared to give better agreement with the test load in the 'unfired' condition.

76. These tests appeared to indicate that quite remarkable economies could be made in the design of cased steel columns, and perhaps the Authors would say whether the above comments represented a reasonable interpretation of the test results.

Mr L. V. Leech (British Constructional Steelwork Association) said that the Paper was a factual report on a very useful piece of research work which had set out to show that the concrete casing of a steel column could not only carry load but could deal with the fire resistance problem as well. From that point of view the actual facts reported did not leave much room for discussion, but there were a number of points to which he wished to draw attention.

78. The very first sentence in the Paper might well have suffered from trimming in order to keep the Paper within prescribed limits. It stated: 'In case of fire it is essential to ensure the integrity of the structure of a building.' That was true, but not absolutely so. It should be qualified by the addition of the words 'for sufficient time to ensure safety of life'—for this was the main requirement of the building regulations—and to permit the fire services to get at the fire without risk of collapse.'

79. In this respect he wished to quote from Mr Eric Bird, an expert in fire protection, who in reply to a question at the Second National Conference of Municipal Building Surveyors, held in November last, had said: 'Only in buildings having an infirmity risk, e.g. hospitals, is fire resistance of more than half an hour necessary to ensure safety of occupants. It is only where evacuation is slow or impossible that you want, as an element of safety, a high structural grade.'

80. Mr Creasy had dealt with the formula which enabled the load-carrying capacity of a column under fire conditions to be designed. The figures used in the formula were based on a design concrete strength of 4000 lb/sq. in., whereas in Table 1 a large number of the test cubes failed to meet that requirement; in fact, one was only 68% of that strength. If the formula could be based on design strengths, so much the better, but he hoped the Authors would comment on these concrete cube tests. After all, the concretes were made under laboratory conditions.

81. Mr Creasy had said so much in favour of the use of concrete encasement that Mr Leach was now reluctant to make the next point, but he felt it was most interesting that in § 50 the Authors concluded that after a short exposure to the heating conditions the cover concrete could not be considered as load-carrying. There was 2 in. of cover on these columns and, if he might draw an analogy, the same must apply to a reinforced-concrete column. If one designed a reinforced-concrete column equivalent to the steel columns tested with their own encasement, one would require about 13 in. square, and if the outer 2 in. of that column were ineffective after being subjected to heat, this would reduce it to 9 in. square. This meant that for load-carrying capacity one lost very nearly 50% of the column.

82. Returning to the encased steel column, surely the fact (if it were a fact) that the cover was not load-carrying after being exposed to heat of the type experienced in a fire was yet another reason for using one of the lightweight encasements, many of which had been developed using naturally heat-resisting materials—e.g. asbestos or vermiculite. These were not load-bearing materials, so that that advantage was lost, but it was very often cheaper to use a little more steel with one of the lightweight

casings, which had other advantages, than to use less steel with a concrete encasement.

83. The question of eccentrically-loaded columns had been mentioned already, and he too felt that this was something the B.S.449 Committee might well reconsider.

84. There were two things concerning fire protection that he wished to bring out. One hour's fire protection was related to the heat derived from 100 000 Btu. In terms of fire load, based on the calorific value of wood and paper, this represented over 12 lb/sq. ft. Investigations carried out on the Continent had shown that the fire load for a modern office building was between 2 and 6 lb/sq. ft, thus showing that the requirements in British Building Regulations were far too onerous. Secondly, in the standard fire test for structural elements the furnace temperature was raised to 700°C in the first 10 min and then progressively to over 900°C after one hour. In a natural fire in a building with a fire load of 6 lb/sq. ft the temperature had been found to rise to a peak of about 830°C in the first 15 min, and because the burning materials were becoming exhausted it dropped steadily to little more than 100°C at the end of an hour. Fig. 14 illustrated this point. One curve was representative of the standard fire-test furnace temperatures, rising to 700°C after 10 min and to over 900°C after an hour. The other curve for the simulated actual fire test, gave 830°C after 15 min and then fairly rapidly dropped down after an hour to a little over 100°C because of the exhaustion of the fuel to feed the fire. It seemed from this that any element standing up to the standard fire test was likely to be more than adequate in a real fire.

FIG. 14

85. Members present had heard about 1 in. concrete providing sufficient protection in a fire but he would go a little further. A Swiss engineer, Dr C. F. Kollbrunner, had written a paper¹⁰ which, translated from the German, was entitled 'Need present-day high steel buildings be afraid of fire?' The author started off with a vociferous 'No' and Mr Leech concluded his contribution by quoting from Dr Kollbrunner to the effect that, from experiments carried out it would be shown that, for contemporary steel structures with fire load of from 2 to 6 lb/sq. ft, the steelwork need no longer be clad. It remained only to convince the various authorities.

Mr A. W. Hill (Deputy Director of Research and Technical Services, Cement and Concrete Association) said that he would like to support Mr Creasy's comments on composite action and the use of lightweight concrete.

87. He differed from Mr Leech on certain points. He felt that there were tremendous advantages in solid protection. It was positive and once there it remained in position. It might suffer slight damage during service but it was very unlikely to have its fire resistance affected by the comparatively light knocks which occurred so easily in a modern building. The reverse was the case with lightweight protections and, in addition, it was all too easy to remove them altogether during structural alterations.

88. However, to return to the Paper, there were three possible omissions. One always found something left out at the end of a research project, and there were obviously good reasons why a project of this sort had to be limited in some respects, but he would have liked to have seen one column at least using limestone aggregate. This material seemed to have been overlooked in the work of the Fire Research Station recently. Perhaps there was some background information which indicated that its early promise was not being maintained. A little more information on that would be useful.

89. It seemed to him, too, that with the tendency today to use high strength steels, at least one unit of this type might have been introduced into the test to give comparable information.

90. The problem in which he was particularly interested was the use of higher strength concretes. In the test under consideration a target figure of about 4000 lb/sq. in. was used. It was possible today, without special techniques, to obtain concrete strengths of 9000, 10 000, and even 12 000 lb/sq. in., and before long this sort of concrete should be in use in Gt Britain. It possibly raised some special problems as far as fire resistance was concerned, and he would be interested to have Mr Malhotra's views. It was a very well-compacted and dense concrete and would have a slightly higher cement content than the type of concrete used in these tests. If the Authors had any further information on the likely fire resistance of this material he would be very pleased to have it.

91. One interesting point he had noticed in the Paper was that for each variable only one test was reported. This, to many who had been dealing with concrete for some years, would appear unusual. Looking at the Paper in detail, however, the results reported seemed to fit very closely to the straight lines on the graph, and perhaps one could assume from this that fire resistance values were very definite, and that there was little experimental variation. Mr Malhotra might have something to add on that point.

92. There were one or two matters on the standard fire test which were of interest and worthy of discussion. In the tests the load was kept at a constant value throughout. In practice this might not be so. The effect of the expansion of the member in a fire, with the various restraints imposed by the adjacent structural units might well cause increased loading on the column. Admittedly, in a standard test one had to define procedure to a large extent, but it seemed to him that the result obtained with this form of testing might therefore be slightly optimistic in relation to the actual behaviour of a column in a building in a fire, especially since in effect no additional factor was introduced on the results. This was not peculiar to the Paper but applied to the whole system of fire testing.

93. The factor of safety aspect had to be considered in relation to the workmanship and the use which was made of buildings in this country. With certain forms of protection and particularly with reinforced concrete and solid protection of steelwork there was very little chance of excessive damage occurring during its life. With some of the other forms of protection this might not be so. Would not an impact test on the specimen before it was placed in the furnace be an advantage?

94. In addition, the normal procedure in fire tests was to round down the period to the nearest hour to give a basic result. In cases where there was a fair margin this could be satisfactory but one could also foresee that a material which received a certificate of one hour resistance on the basis of a test of 1 h 3 min or 1 h 5 min having substantially less than that value in service, because of these defects.

Mr G. M. J. Williams (Scott & Wilson, Kirkpatrick & Partners) said that he was pleased to see this Paper in the Institution's *Proceedings* because it gave the opportunity to raise a question which had worried him for some little time. Indeed, possibly the Authors were aware of the point that he proposed to raise, because in § 60 they said that more basic information was required on the properties of concretes at high temperatures and the influence of stanchion size. Apparently most of the rules of fire resistance were based on tests on 8-in. $\times$ 6-in. $\times$ 35-lb steel stanchions, and it was assumed that any other size of section would require the same thickness of protection to achieve a similar degree of fire resistance. This had always seemed to him a startling coincidence and he was rather surprised that the Authors did not justify it. They just stated it as a fact and the reader was expected to believe it. For his part he would have liked to see some evidence for it. The Authors might know of some but it was not readily accessible to the ordinary engineer.

96. The range of standard sections available for columns went from about 10 lb/foot run to over 400 lb/foot run, and, of course, much heavier compounds could be made. It was reasonable to assume as a very rough criterion that if there were a joist section inside a concrete casing, the rate of temperature rise for a given difference of temperature between the steel and the exterior was proportional to the perimeter at the average thickness of the cover, which one might call P , because that gave the area through which the heat flowed, and, inversely proportional to the thermal capacity of the core of the column, namely, the mass times the specific heat of the steel together with the mass times the specific heat of the concrete in the core, which he would call W (Fig. 15). In other words, at any given moment in the fire test the rate of increase in temperature was roughly proportional to that factor for any given column; and, using the information from the Authors' tests, he had roughly calculated the time at which various typical columns might be expected to reach a softening temperature. The results could be tabulated as shown in Table 7. The Authors' 35-lb column, in the units he used, had a ratio of P/W of 3, and, taking as a typical example their column with 2-in. cover, it had about three hours fire resistance in the test. Taking

FIG. 15: RELATIONSHIP BETWEEN THE RATE OF TEMPERATURE RISE AND THE SIZE OF COLUMN WITH A GIVEN FIRE PROTECTION

$$d\theta_1/dt \propto P/W(\theta_0 - \theta_1)$$

the 320 lb/foot column-core section, and again assuming this 2-in. cover around it, P/W was approximately 1 in. in the same units, so that for a given temperature difference between the furnace and the core of the column, the temperature of the core was increasing at about a third of the rate of that of the column that the Authors tested; and he had roughly worked out that it would take 6–7 h to reach the collapse temperature.

TABLE 7: ESTIMATED FIRE RESISTANCE OF COLUMNS OF VARIOUS SIZES WITH THE SAME THICKNESS OF PROTECTION

Weight of steel section	8 lb/ft	35 lb/ft	320 lb/ft
P/W	24	3	1
Estimated resistance period	70 min	3 h	6–7 h

97. Thus it appeared that the current regulations were conservative for the very heavy columns; and, of course this related to Mr Leech's remarks, because as he had said, a certain fire load was needed to produce enough heat to raise a given amount of steel through a given temperature, and it was difficult to conceive where in an ordinary building which was not used for storage purposes enough heat could be found to raise the very large columns to softening point. But perhaps it was most significant when they went the other way—down the scale towards the very light columns. At the moment there was a lot of emphasis on standardized types of construction that used very light mild steel columns. Quite frequently the cold-pressed sections were used, usually in the form of box columns. They had been used for schools for many years, but there was a pressure to extend them to multi-storey buildings for universities and hospitals. There were buildings in progress with this type of construction going up to four storeys.

98. A typical column of this type, made of two cold-pressed channels put together to make a box column, might weigh 8 lb/ft. If the fire resistance around that column consisted of 2-in. cover of concrete, his criterion of P/W would come to 24, so that for a given temperature difference between the inside and the outside of that column, the temperature of the steel core would increase at eight times the rate of the column the Authors tested. On that basis he had calculated that the column, with 2 in. of fire protection, would fail in about 70 min. It seemed to him that fire regulations should take some account of the size of the section involved, and he was very interested to know whether the Authors had considered this point and whether they knew of anything to confirm what he had said. This was pure hypothesis but it seemed to him that it would not be very difficult for the Authors to do a more refined calculation of the time of collapse on the lines described in the Paper.

Mr J. W. Dougill (Imperial College of Science and Technology) said that Mr Malhotra, in his introduction, had mentioned the work being done at Imperial College in Professor Baker's section on spalling of concrete. The work was sponsored by the Fire Service Research and Training Trust.

100. It appeared that spalling of gravel concrete could occur in three separate ways. These were surface failures, separation of external corners, and aggregate splitting.

101. The problem of aggregate splitting had been examined and it seemed that for concrete mixes, using the same type of aggregate, increases in the stiffness or the strength of the mortar could increase the likelihood of splitting of the aggregate.

Also, an increase in the maximum size of the aggregate increased the likelihood of splitting. Some tests would be carried out this summer to check these tentative conclusions.

102. It appeared that the separation of the external corners of the concrete was due to a tensile stress component across the corners similar to the radial component obtained when the surface of a cylindrical column was heated. Concrete was weak in tension and this type of failure was, in fact, almost inevitable.

103. The other form of spalling examined at Imperial College was where parts of the surface of a large concrete panel disintegrated violently and pieces were thrown off. At a later stage of heating this might become more gradual, and although progressive the concrete tended to adhere and sloughed off slowly.

104. It was believed that the explosive form of failure was caused by the formation of a number of cracks parallel to the surface of the concrete but just within the surface.

105. The concrete outside the crack acted as a thin plate under a loading, in its own plane, which increased with increase in temperature. Failure occurred by buckling outwards when the critical load on the plate was exceeded.

106. Mr Dougill then showed a number of slides in which the different forms of spalling could be distinguished.

107. Using simple criteria for the fracture strength of concrete it had been possible to explain the effects of load and generated pore pressure on the occurrence of spalling, and some tests would be done in the near future to check these conclusions.

108. Referring to the main topic, the encased stanchion, he was interested in equation (1), § 51, in which failure was based on yield criteria for both the concrete and the steel.

109. Both steel and concrete, however, lost strength and elasticity when heated. It was possible that the strength and fire resistance of a long column when exposed to fire might be determined more by stability than yield strength. This would be particularly so if the elasticity or Young's Modulus were reduced, at a far greater rate than the yield strength, on heating. If this were so equation (1) would be unsatisfactory.

Mr D. I. Lawson (Director, Fire Research Station) referred to Fig. 12 in the Paper, where the calculated curve of the ultimate load did not seem to vary for a time of exposure between half an hour up to nearly two hours. At first sight this seemed a little remarkable, for as things were exposed to fire their temperature must rise, both on the surface and on the layers beneath the surface, and both concrete and steel would lose strength as the temperature rose. Taking these factors together it seemed a little odd at first sight that one should get a horizontal part of the curve in which the ultimate load was independent of the time of exposure. He was sure there was a physical explanation for this but he could not see it himself at first sight.

Mr R. A. Shewring (Associate Partner, A. J. & J. D. Harris) referred to § 62 which ended with the statement 'For fire resistance periods of less than 2 h a thickness of 1 in. is stipulated which, though in excess of the amount necessary from fire considerations, is considered to be the minimum practical cover', and questioned whether even 1 in. was a practical minimum. He recalled that when he had been with a firm of contractors, who had done a lot of casing of steel frames, there had been difficulty in properly consolidating only 1 in. of cover, particularly over the flanges of stanchions and beams. As a result it had been found well worth while to increase the cover over flanges to 2 in. minimum to ease the concreting of the encasement. He wondered whether there were any contractors present who could give their views on this point.

The following contributions were received in writing:

Mr C. Hobbs (John Laing Research and Development Ltd) observed that in § 50

the Authors stated that 'it would appear unlikely that the majority of the concrete cover could be considered as carrying load after a short exposure to the heating conditions'. Yet separate tests of reinforced lightweight concrete columns had shown that even after a 2-h test only about the outer $\frac{1}{2}$ in. of the concrete was really affected. This would seem to suggest that in the case of a lightweight aggregate concrete casing a substantial part of the concrete cover could continue to carry a load for a prolonged period under serious fire conditions. Could the Authors comment upon this point?

113. The whole of the work was also carried out upon concrete casings which were reinforced in accordance with the recommendations of B.S.449, 1959, or in the case of columns with 1-in. cover with a 6-in. sq. light mesh of 12 s.w.g. steel. The Authors concluded that the purpose of this reinforcement was merely to hold the concrete in position so that it continued to afford protection to the steel, and that the lighter reinforcement was completely adequate. But in other work on reinforced lightweight aggregate concrete columns it had been shown that mesh reinforcement of this kind was not necessary at all, owing to the freedom from spalling of these concretes. Did the Authors consider that any mesh reinforcement of this kind was therefore necessary with lightweight aggregate concrete claddings?

Mr P. B. R. Johnson (Engineer, George Wimpey & Co. Ltd) made the following comments.

Physical properties

115. Referring to § 40 it was agreed that for a fuller understanding of the performance of the stanchions it was necessary to have a knowledge of the properties of steel and concrete at high temperatures. It was felt that the splendid experimental results could be better evaluated if the Authors would produce supplementary information on these properties, which were also essential for the development of any theory or design formula.

(a) Steel

116. It was implied that the stanchions were of steel to B.S.15 but B.S.449 also covered other steels and it was hoped that the new tests referred to in § 40 would cover this point.

117. In § 18 the modulus of elasticity was stated, but this could be plotted with advantage over the 0–1200°C range and probably would reduce by about half at the higher temperature.

118. A plot of the increase of specific heat over the 0–1200°C range would assist in evaluating § 58.

119. A plot of the decrease in thermal conductivity over the 0–1200°C range would be useful in applying experimental results to stanchions which were interconnected with other steel members.

120. A plot of creep over the 0–1200°C range might be significant, and a plot of the increase in the coefficient of expansion over this range would be useful in evaluating § 34, etc.

121. A plot of yield and ultimate stress for various steels to B.S.449 showing the variation both with rate of loading and increased temperature would be most valuable. It was not thought that the results of Lea⁷ which applied to early 1920 tests on mild steel reinforcing bars were appropriate. Until the new B.R.S. test results were obtainable it was suggested that a comparison could be made with the results of Manjoine¹¹ for American mild steel using an appropriate rate of loading.

(b) Concrete

122. It was suggested that similar information on the relevant physical properties of various types of concrete was needed over the 0–1200°C range.

123. While a 0–1200°C, the 6-h standard fire temperature, had been suggested as a range for plotting properties, it was appreciated that 0–700°C would be adequate for practical use in the present connexion.

Method of testing (F series stanchions)

124. B.S.476, clause 10c required that the restraint applied to the test element should be similar to that applied to the element in service. In § 26 the Authors said that the stanchions were allowed to expand during test. There are cases in practice where stanchions were not fully free to expand. Would it not have been desirable to test some stanchions under conditions of total restraint as was done for 'C 18' in a previous series of tests?² The interaction of the benefits of restraint on the casing⁸ with the disadvantage of increased stanchion load (§ 56) due to the prevention of expansion would seem of interest.

Desirable mix for casings

125. B.S.449 called for a 28-day works strength of 3000 lb/sq. in. for casings. Reference 8 showed that leaner mixes were preferable to stronger ones at high temperatures. Did the Authors consider the mix chosen for type G tests of 1:2.7:4.5 a desirable one to use in practice?

Casings generally

126. Recent papers on fire resistance of cased steelwork seemed to have concentrated on the insulation value of the casing in keeping steel temperatures low. Observations of uncased steel after fires showed that failure was generally by lateral-torsional bending. All the stanchions in the present series of tests had casings the strength of which at increased temperatures might have had advantages in resisting such bending. Such advantages would not be present in casings which just provide insulation. Would the Authors like to comment on this property of casings?

Mr D. E. Dobson (Pretoria, S. Africa) wrote that the Authors were to be congratulated on having carried through such important research work and also for having set out the results of it so clearly for others. Their proposals for applying this research so as to alter B.S.449 and to amend the Model Building By-Laws were sound and were supported. Because the Model Building By-Laws would eventually become mandatory National Building Regulations the application of the Authors' work in the field became of even greater importance.

128. The load factor of 5.57 for the eccentrically-loaded column, as against 2.78 for the axially-loaded one, suggested, as the Authors had pointed out in § 57, that B.S.449 might be relaxed to allow for a greater working load. If this was not done one might consider a relaxation in the required fire resistance rating for an external column; this was apparently being urged on other grounds anyway. The Authors' views on this alternative would be welcome.

129. Actually the whole field of application of fire research findings in building by-laws or regulations was in need of review by an authoritative committee. Mr Dobson had in mind something similar to that important report 'Fire grading in buildings', published as Post War Building Studies No. 20 (H.M.S.O., London, 1946).

130. Such a committee, for instance, could integrate, or at least put into perspective, the work on the European continent on structural fire protection, in comparison with corresponding British thought: the Continental work was exemplified by Kolbrunner and Boué's 'Method of calculation for fireproofing of steel-framed structures and its practical application'.

131. The committee could also consider to what extent a conservative approach was necessary to required fire resistance ratings in order to provide some useful degree of protection in time of emergency such as nuclear attack, when there might not be an effective fire service in many areas.

TABLE 8: A SUPPLEMENT TO TABLE 5

Column No.	Area of stanchion, A_s	Mean steel temperature	Yield stress, f_t	Load on steel, $A_s f_t$	Area of concrete, A_o	Mean temperature of concrete assumed	Concrete strength, U_t	Load on concrete, $A_o U_t$	Estimated ultimate load	Observed ultimate load
	sq. in.	°C	ton/sq. in.	tons	sq. in.	°C	lb/sq. in.	tons	tons	tons
F1G	10	620	4.4	44	48	530	1750	37.5	81.5	87
F2G	10	600	5.1	51	48	490	2160	46.3	97.3	104
F3G	10	551	6.6	66	48	410	2770	59.3	125.3	122

TABLE 9

Column No.	Load on steel $A_s f_t$	Observed ultimate load	Load unaccounted for; assumed carried by concrete L_o	Useful concrete area, $A_o = L_o \div U_t$	Actual area of concrete, A_{cr}	Proportion of concrete strength or area utilized, $j = A_o \div A_{cr}$
	tons	tons	tons	sq. in.	sq. in.	
F1G	44	87	43	55.0	70	0.785
F2G	51	104	53	54.9	110	0.500
F3G	66	122	56	45.3	158	0.287

132. Returning to the Authors' researches, there was one point on which more work was needed before their contention in § 52 could be accepted. The Authors had said, in effect, that in the formula for estimated ultimate load of a heated column, $W = A_s f'_t + A_c u'_t$, it was sufficient in arriving at the load carried by the concrete casing to assume the area of concrete, A_c , effectively involved to be equivalent to BD , the enclosed area of the stanchion. This assumption might eventually be justified, but one could not make it only on the basis of the evidence presented by the Authors in Table 5, for all four tests given there were on the same sized stanchion and with the same concrete cover. To test the Authors' hypothesis, results were needed on different sized stanchions and with different thicknesses of cover.

133. The Authors were, of course, using Table 5 to illustrate something else, and so did not use all the comparative results available to them. They would also perhaps have additional results available by now, and it would be interesting to know how Mr Dobson's suggested alternatives for predicting the ultimate load accorded.

134. Using then the information given in Tables 4 and 5, and assuming certain average temperatures in the concrete cover, Mr Dobson had tried to check the Authors' hypothesis (that BD was the best estimate of A_c) for the three columns F1G, F2G, and F3G, which have a concrete cover of 1, 2, and 3 in. respectively. The results were shown in Table 8, which was a supplement to Table 5 and was given in the same form as that table for ease of reference. The correspondence of estimated with observed ultimate load was not as good as that shown in Table 5 and led one to wonder what it would be like with stanchions of different size. Perhaps there was a better estimate of A_c than BD .

135. Actually the load carried by the concrete casing L_c could be estimated directly. This had been done in Table 9 for the same three steel columns F1G, F2G, and F3G, and worked out at 43, 53, and 56 tons respectively. This information had then been, used to illustrate two other hypotheses as to the best estimate of L_c :

- (a) given by column 4 of Table 9, that to ascertain how much concrete is actually being stressed (i.e. to find A_c) one needed only to divide the load carried by the concrete by its crushing strength at the average temperature concerned. The results of this calculation, viz. 55, 54.9, and 45.3 sq. in. suggested that if a constant figure were to be taken for an 8 in. $\times$ 6 in. R.S.J. then it should rather be 52 than 48 sq. in. But a different figure would probably be needed for the differently sized stanchions. Mr Dobson suggested, however, that the matter should be generalized, and that the best estimate of A_c would be in the form $A_c = BD + kB$; and
- (b) rather than working with A_c , the useful concrete area, derived indirectly, that one should work directly with a proportion of the real concrete area A_{cr} , i.e. with jA_{cr} . A_{cr} had been given in column 5 of Table 9 and A_c , as found above, had been used to obtain an estimate of the sort of figures one could expect for j , where jA_{cr} was to replace A_c in the ultimate strength formula. Further work would be needed to ascertain whether j was in fact predominantly a function of the cover, of the proportions of the stanchions, or of both.

136. To summarize these suggestions:

$$W = A_s f'_t + BD U'_t$$

might be better as

$$W = A_s f'_t + B(D + k) U'_t$$

or

$$W = A_s f'_t + jA_{cr} U'_t$$

Mr A. K. Chattopadhyay (Freeman, Fox & Partners), congratulated the Authors on their stimulating Paper and declared himself to be in sympathy with their general aim.

138. The subject was a matter of the science and behaviour of materials, and should be taken from a scientific approach and with sound judgment of the experience of test results. At the moment the data available were not sufficient to make any definite conclusion. The tests were done on a particular rolled mild steel I-section of the steel components, and the variables did not include different sizes and shapes or the different qualities and grades of steel materials. Though the effects of these variables were unpredictable, most likely a variable thickness of the protective cover could be related to the cross-sectional dimensions and the grades and qualities of the steel components. The existing fire regulations required to adopt more realistic figures for 'fire-load', keeping in mind the fact that fire could only develop with inflammable materials. The actual 'fire-resistance times' needed to be assessed, beside taking into account size, purpose, location of the premises, together with the latest available means of giving alarm signals, number and location of fire extinguishers, provisions for evacuating the building, the time required for the arrival of the fire service, etc. The various buildings then could be classified according to their behaviours. For example, hospitals which needed longer evacuating time should be treated differently from a block of offices containing steel furniture and few inflammable materials.

139. On the aforesaid basis, a careful assessment should be made of the means of protection and this should only be provided if it was shown to be necessary. Some uncased steel stanchions had recently been used in France. There, as elsewhere, the authorities in this field were considering relaxation of the regulations by reducing the 'fire-resistance times' for ordinary buildings of less than 28 metres (92 feet approx.) in height, for example, from 1, 2, 3, and 4 h to $\frac{1}{2}$, 1, and $1\frac{1}{2}$ h, according to the category of building, as defined by the number of occupants and by the number of storeys.

140. When all these data were available, new rules could be produced which would permit further economy of materials combined with greater safety. As had been proposed by the Authors, however, in § 63 of the Paper, the rules if adopted might be regarded as tentative until further research results became available. It seemed certain that more ambitious tests would have to be carried out to achieve the desired objectives.

The Authors, in reply, wished to thank the contributors for the stimulating and informative comments on the Paper.

142. Perhaps one of the most important features of the tests was the very clear manner in which the strength of concrete-encased stanchions had been shown to be the sum of the strengths of the steel and the concrete, both in structural tests and during exposure to conditions of fire.

143. The Authors agreed with Mr Creasy on the importance of that point, and looked forward to a future revision of the design clauses in B.S.449 relating to encased stanchions.

144. Mr Leach and Mr Johnson had raised questions about the concrete used in the programme. The crushing strengths of the concretes shown in Table 1 were the mean of six specimens. The concretes were designed to have a crushing strength of 4000 lb/sq. in. after 28 days of storage in water, and the mean of the cube strengths for each stanchion had an average value of 3870 lb/sq. in., with a standard deviation of about 300 lb/sq. in. The different strengths of the air-stored cubes were due to the uncontrolled temperature and humidity conditions in the testing laboratory. Previous experience had shown that the strength of large concrete elements could be related to the strength of water-stored cubes, as little drying took place in 28 days.

145. The mix proportions used for the laboratory specimens would be unlikely to be used in practice, because greater control of the concrete could be exercised in a laboratory. The specified works cube-strength was a minimum requirement, and

because of greater variations in concrete on site a richer mix would have to be used to achieve that minimum strength.

146. In reply to points made by Mr Leach and Mr Chattopahdhay on the fire resistance requirements in building regulations, the Authors suggested that the requirement for fire resistance in a building and the relationship between fire load and the standard time/temperature curve were outside the scope of their Paper. While it was true to say that Acts under which the building regulations were formulated were primarily concerned with the safety of life, the requirements usually specified ensured that the buildings were not likely to become structurally unsafe in case of fire.

147. The example shown in Fig. 14 of temperatures achieved in an experimental fire was relevant to that particular test only and had no universal application. The course of a fire was determined by a number of factors beside the fire load, such as disposition of the contents, geometry of the room, and available ventilation. In a recent full-scale fire test in a building with a fire load of 5 lb/sq. ft, higher temperatures than were shown in Fig. 14 had been obtained for longer periods.

148. For an objective assessment of the performance of structural steel in building fires a research programme was being conducted at the Fire Research Station, where the effect of factors such as fire load, size of section, its location in a building and available ventilation were being studied.

149. There were many examples of unprotected steelwork leading to structural collapse in a fire and the need for protection was fully appreciated. The Paper had not rejected the use of lightweight encasements for that purpose. The test data had shown beyond doubt that a concrete encasement not only had an insulating function but also contributed significantly to the structural strength of the stanchions; in the case of specimen F2GU (Table 5), the concrete was carrying a greater load at failure than the steel section.

150. Mr Hill had asked about the omission of limestone aggregates and high tensile steel from the test programme, which had to be limited to enable its completion in the given time. It was thought that the better integrity at high temperature of limestone concrete would not have made a substantial difference in view of the presence of reinforcement in the concrete cover. The performance of modern high strength steels was being examined in current research programmes.

151. The Authors were interested in the use of the high-strength concrete mentioned and looked forward to an opportunity of testing such structural elements.

152. Fire resistance tests were time-consuming and expensive and hence not many replicates were made. Whenever repeat tests had been performed, the experimental variation in results had been found to be very small.

153. The maintenance of constant load during a fire test enabled the performance of different types of structural element to be compared on a standard basis. While it was true that in practice the load would alter depending upon the degree of restraint, it had not been found possible to specify that restraint.

154. The Authors agreed with Mr Williams that the question of size of stanchion section was one of importance. It was known that stanchions larger than 8 in. $\times$ 6 in. would, for the same cover, possess greater fire resistance, but there was not as yet enough information to allow for this in design. It was an over-simplification to suggest that the rate of temperature rise in a stanchion was proportional to the perimeter, the least lateral dimension played an important part in fire resistance. As had been pointed out in the Paper, the Authors hoped, with further knowledge of the properties of steel and concrete, to predict the fire resistance of a structural element in terms of its size and shape as well as its specific heat. When dealing with small sections of the type used in system buildings, consideration was given to the size of section in assessing the protection required.

155. The Authors were interested to hear Mr Dougall's remarks on spalling of concrete and looked forward to studying the complete investigation.

156. Mr Dougall had raised a valid point in suggesting that the value of Young's

modulus for concrete was reduced with increase in temperature. Reductions of about 25% have been reported at about 300°C.

157. In reply to Mr Lawson, the Authors pointed out that changes in the properties of steel and concrete with increase of temperature were extremely interesting. After the initial loss in strength of the concrete cover, which had been estimated to occur in about 30 min. in Fig. 12, the maximum temperatures in the steel and load-bearing concrete were about 100°C. The temperatures then steadily increased during the test. The strengths of concrete and steel did not reduce appreciably until 300 and 400°C respectively, hence for a certain period the strength of the stanchion was not immediately reduced.

158. The Authors agreed with Mr Johnson on the need for further data on the properties of steel and concrete at elevated temperatures.

159. Until the strength of concrete made with lightweight aggregates had been investigated at high temperatures it would not be possible to comment on Mr Hobbs' remarks. It would seem that lightweight aggregate concrete showed lower reduction in strength than concretes made with gravel aggregates, though at temperatures in excess of 600°C it could not be expected to retain more than a small proportion of its original strength. Its lower thermal conductivity ensured that the temperatures inside the section were lower than in the corresponding dense concrete section. It was possible that light reinforcement as used for specimen F2GU might not be necessary for the lightweight concretes; a test would be necessary to confirm that.

160. The Authors agreed with Mr Dobson on the need to extend the tests to include different sizes of stanchion and different thicknesses of cover.

161. With regard to the validity of assuming that the effective area of concrete in equation (1) could be represented by $B \times D$ for the encased stanchions tested, the simple representation had been derived from consideration of the results of all the encased stanchions tested not just those with a cover of 2 in. In fact the effective area of concrete which might be considered as load-bearing would increase as the thickness of cover increased, but for practical purposes when the cover ranged between 1 in. and 3 in. the difference between the effective area and $B \times D$ was slight.

162. The Authors disagreed both with the values of mean temperature of concrete in Table 8 and with the values of effective concrete area in Table 9.

163. The behaviour of external columns in case of fire was under investigation at the Fire Research Station and was expected to establish a relationship between the performance in the standard fire resistance tests and when exposed to actual fire conditions of varying severity.

164. One of the points raised by Mr Chattopahdhy covered a much wider field of building regulations and utilization of data from fire tests than it was possible to deal with in Discussion. The factors he had mentioned were taken into account when building regulations were drafted in Gt Britain and the fire resistance requirements for structural elements applied equally to steel and to other materials.

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