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N. S. Pearce and W. W. Stanzak

Load and Fire Test Data on Steel-Supported Floor Assemblies

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Special Technical Publication No. 422, 1967, (p. 5 — 20)

Research Paper No. 344 of the Division of Building Research
Ottawa, December 1967

DONNEES OBTENUES AU COURS D'ESSAIS DE RESISTANCE AU FEU D'ELEMENTS COMPLEXES DE PLANCHER A OSSATURE D'ACIER

SOMMAIRE

Les auteurs présentent les données recueillies grâce à des essais normaux de chargement et de résistance au feu réalisés sur onze ouvrages à ossature d'acier. Ils élaborent une méthode d'analyse des déformations élastiques que subirait une poutre mixte idéale en acier et béton et ils proposent une définition relative au niveau de résistance combinée que manifestent les échantillons d'essai. Ils étudient les sollicitations que la charpente d'essai devra supporter et leur importance pour la résistance combinée qu'elle devra exercer durant l'essai de résistance au feu.

Le niveau de résistance combinée est calculé pour un certain nombre de types d'éléments complexes et on expose brièvement le mécanisme de la rupture sous charge pendant l'essai de résistance au feu. A la lumière de ces renseignements, les auteurs étudient ensuite quelques aspects des méthodes actuelles d'essais de résistance au feu et ils proposent certaines améliorations qu'il serait possible de réaliser.

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Load and Fire Test Data on Steel-Supported Floor Assemblies

REFERENCE: N. S. Pearce and W. W. Stanzak, "Load and Fire Test Data on Steel-Supported Floor Assemblies," *Symposium on Fire Test Methods—Restraint & Smoke 1966, ASTM STP 422*, Am. Soc. Testing Mats., 1967, p. 5.

ABSTRACT: Information gained from standard load and fire tests on eleven steel-supported constructions is presented. An elastic analysis of an idealized composite steel-concrete beam is developed, and a definition for the degree of composite action inherent in test specimens is proposed. The restraint afforded by the test frame and its importance to continued composite action during the fire test are examined. The degree of composite action is calculated for a number of assemblies, and the mechanism of load failure during fire test is discussed briefly. Using this information, some aspects of present fire test practice are examined and possible improvements are suggested.

KEY WORDS: fire tests, steel construction, reinforced concrete, failure, thermal expansion, restraint

Nomenclature

- A_s Cross-sectional area of steel beam
- A_t Transformed cross-sectional area of composite beam
- b Equivalent effective width of concrete slab
- C Degree of composite action
- d Effective depth of concrete slab
- d_s Distance from top of slab to centroid of steel beam
- D Depth of steel beam
- h Overall depth of structural system
- $\bar{h}$ Height of neutral axis from lower flange of steel beam
- $\bar{h}_o$ Height of neutral axis from lower flange of beam-floor assembly
- I Moment of inertia of beam-floor assembly
- I_d Moment of inertia of floor

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² Steel industries fellow, Division of Building Research, National Research Council, Ottawa, Ont., Canada.

I_o	Moment of inertia of steel beam and floor without shear connection
I_s	Moment of inertia of steel beam
I_t	Moment of inertia of composite beam with full shear connection
l	Effective span of beam
n	Ratio of modulus of elasticity of steel to concrete
w	Web thickness of steel beam
Y	Distance from top of slab to neutral axis of composite beam
σ_b	Extreme fiber stress at bottom of steel beam
σ'_t	Extreme fiber stress at top of steel beam
σ_t	Stress at top of concrete slab
M	Live load bending moment

This paper is concerned with loading and restraint as it affects the structural performance of steel-supported floor constructions during a fire test. The applied loading on these structures during the fire test has been a matter for concern since fire testing began in North America.

The 1918 edition of the "ASTM Standard for Fire Tests of Building Construction and Materials," which in those days went under the designation ASTM C 19 – 18, contained the following paragraph:

The floor or roof shall be loaded in a manner to develop in each member of the construction, stresses equal to the maximum safe working stress allowed in the material of the members.

By the year 1926, the paragraph had been modified by the addition of wording which was intended to take into account the apparent impracticability of fulfilling these loading requirements. The applicable paragraph of the standard, then numbered ASTM C 19 – 26 T, read as follows:

During the fire endurance and fire stream tests, the construction shall be loaded in a manner calculated to develop theoretically, as nearly as practicable, the working stress in each member contemplated by the design.

The current edition of the standard (E 119 – 61) contains wording substantially similar to this, except that reference to the fire stream test has been deleted, since this is no longer a mandatory requirement of the standard, and the load is now specifically referred to as a superimposed load.

Discussion within ASTM Committee E 5 which is charged with the responsibility for the standard has indicated that there is again a need to revise the test method or interpretation of the standard, bearing in mind the manner in which this information is applied. There is an increasing tendency to employ information derived from fire tests out of the context of the specific assemblies tested. This is particularly true of the practice whereby beam classifications obtained from constructions which are tested as floor and ceiling assemblies are applied to structural members which are

in no way representative of the tested assembly. It should be noted that although assemblies may appear to have basic structural similarities and similar protection, their behavior under identical temperature conditions in the standard fire test varies considerably. One reason for such variation is thought to be the degree of composite action inherent in the construction of such assemblies. A second reason may be the variation in restraint afforded by the attachment of the floor to the support beam.

Various groups participating in the work of Committee E 5 have been active in acquiring information intended to provide for greater understanding of these problems. This paper presents the results of a study based on a series of fire tests on floor and ceiling assemblies conducted in accordance with the present requirements of the standard. The purpose of the investigation was:

1. To study the degree of composite action inherent in beam-floor assemblies commonly submitted for fire test.
2. To investigate the degree of restraint provided by the test frame to steel beams incorporated with various types of floor assemblies during fire tests.
3. To examine the influence of (1) and (2) above on the structural performance of an assembly during fire tests.

To this end, measurements were made of:

- (a) Live load strains in the critical fibers of the beam during and subsequent to the application of the load.
- (b) The strain on the frame during the fire test.
- (c) Deflections and beam temperatures throughout the test.

Apparatus

Floor Furnace—The equipment used by Underwriters' Laboratories of Canada for "Fire Tests of Floor or Roof and Ceiling Constructions" consists of a test furnace designed to evaluate the performance of these constructions under fire conditions in accordance with the requirements contained in ASTM Methods E 119. The restraining frame, within the confines of which the tested assemblies are erected, consists of a welded structural steel assembly, having inside dimensions of approximately 18 by 14 ft. The heated volume beneath the restraining frame of approximately 900 ft³ is enclosed by a dry wall concrete block construction protected with vermiculite plaster. An environment following the standard time-temperature curve is produced by the controlled output of 84 natural gas burners arranged in a matrix at the bottom of the furnace and supplied with combustion air through ports located below this level.

Loading System—The hydraulically operated loading equipment is designed to simulate the bending moments resulting from a uniformly distributed load applied over the floor and a linear uniform load over the support member(s). The load is applied by hydraulic jacks, mounted

on five separate loading units, which span the test assembly, with each loading unit controlled through a separate channel. The jacking arrangement provides for a total deflection at the center of the assembly of approximately 18 in. The system is designed to provide for floor loadings of up to 450 lb/ft² with a maximum superimposed linear load along the support beam of 3000 lb/ft.

Deflection Gages—Floor deflections are indicated by vernier measuring tapes connected through a piano wire and pulley system to small counterweights located at significant locations on the surface of the assembly. The measuring tapes are calibrated to read in increments of $\frac{1}{100}$ in.

Temperature Recording—Furnace temperatures are measured by chromel-alumel thermocouples, unexposed surface temperature by iron-constantan thermocouples, arranged in accordance with the requirements of the standard and displayed on a strip chart recorder.

Strain Gages—The strains are measured by metal film, epoxy-backed, temperature compensated, strain gages having a gage length of $\frac{1}{4}$ in. and a resistance of 120 ohms. The gages are connected in a quarter bridge, external dummy, three-wire configuration.

Strain Indication—The read-out devices used with the strain gages are of two types: (1) a bridge amplifier and meter and (2) a digital strain indicator.

Test Assemblies

Assembly 1—Structural steel beam, supporting equal spans of cellular steel floor units, 3 in. deep, with 2½-in. concrete topping. Protected by suspended membrane ceiling.

Assembly 2—Structural steel beam, supporting open web steel joists on 2-ft centers and arranged in spans of 10 ft and 6¾ ft. Concrete topping, average depth 3¾ in., placed on ribbed metal lath reinforced by wire fabric mesh. Protected by suspended membrane ceiling.

Assembly 3—Structural steel beam, supporting equal spans of cellular steel floor units, 3 in. deep, with 2½-in. concrete topping. Protected by sprayed fiber.

Assembly 4—Three structural steel beams, each supporting 5-ft wide sections of cellular steel floor units 1½ in. deep, with 2½-in. concrete topping. Protected by sprayed fiber.

Assembly 5—Structural steel beam, supporting equal spans of steel floor units 1½ in. deep, arranged in a blend of cellular and fluted units with 2½ in. of concrete topping. Protected by suspended membrane ceiling.

Assembly 6—Structural steel beam, supporting a blend system of fluted and cellular (Hi-Bond) steel floor units 3 in. deep, arranged in spans of 10 ft and 6¾ ft, with 2½-in. concrete topping. Protected by sprayed fiber.

Assembly 7—Structural steel beam, supporting a blend system of fluted and cellular (Hi-Bond) steel floor units, 1½ in. deep, arranged in equal spans, with 2½-in. concrete topping. Protected by sprayed fiber.

Assembly 8—Structural steel beam, supporting equal spans of blended, fluted, and cellular steel floor units, 1½ in. deep with 2½-in. concrete topping. Protected by suspended membrane ceiling.

Assembly 9—Structural steel beam supporting a blend system of fluted and cellular steel floor units, 1½ in. deep arranged in spans of 10 ft and 6¾ ft, with 2½-in. concrete topping. Protected by sprayed fiber.

FIG. 1—Location of strain gages on supporting steel members.

FIG. 2—Location of strain gages on restraining frame.

Assembly 10—Structural steel beam, supporting open-web steel joists on 2-ft centers and arranged in equal spans. Concrete topping, average depth 3¼ in., placed on ribbed metal lath reinforced by wire fabric mesh. Protected by suspended membrane ceiling.

*Assembly 11*³—A simple beam of rectangular cross section, with one end resting on a bearing plate and the other on rollers protected with ½-in. asbestos fiber board, and the furnace opening was closed with an unattached insulating topping consisting of ⅜ in. of asbestos fiber board, ⅝-in. asbestos millboard and 6 in. of rockwool approximately 3 ft wide.

The intermediate support beams in each of the assemblies briefly described above (except Assembly 11) had an effective span of approximately 160 in. and were restrained against longitudinal expansion. The

³ Test conducted at NRC Division of Building Research.

TABLE 1—Composite *Nature of the Assemblies.*

Assembly No.	Beam Type	I_s , in. ⁴	Average Concrete Strength, psi	n	Live Load Bending Moment, in. · lb	Design Live Load Stress, psi	Measured Stress, psi		I_o , in. ⁴	I_t , in. ⁴	$\bar{h}_o$, in.	$C = \frac{I - I_o}{I_t - I_o}$
							Upper Flange	Lower Flange				
1.....	10 WF 25 A-36	133.2	2 808	11	464 000	17 580	9 960	13 140	138	489	5.7	0.179
2.....	8 WF 31 A-36	109.7	2 863	10	502 000	18 300	14 460	15 300	115	365	4.12	0.080
3.....	10 S 25.4 A-7	122.1	3 570	8	375 000	15 350	5 490	9 720	129	557	6.4	0.276
4.....	8 WF 24 A-36	82.5	3 140	10	389 000	18 720	5 500	13 170	88	276	5.65	0.421
5.....	8 WF 17 A-36	56.4	2 950	10	185 000	14 000	900	6 255	62	215	7.0	0.947
6.....	10 WF 25 A-36	133.2	3 000	10	466 000	17 650	3 000	10 050	138	499	7.7	0.604
7.....	10 WF 25 A-7	133.2	3 100	10	428 000	16 240	0	5 160	138	410	...	1.00
8.....	10 WF 21 A-36	106.3	2 600	12	359 000	16 700	4 230	10 140	111	325	7.05	0.650
9.....	10 WF 33 A-36	170.9	4 346	7	646 000	18 470	5 360	10 820	176	516	6.52	0.627
10.....	8 WF 35 A-36	126.5	2 357	13	502 000	18 750	13 620	14 040	114	333	4.10	0.146
11.....	2 by 4 Rect A-7	10.7	...	...	...	20 000	...	...	...	...	...	0

^a Taken as 1/4.

beam of Assembly 11 had an effective span of approximately 186 in. and was completely unrestrained. Strain gages were attached to the top flange, web and bottom flange at the center of the supporting structural steel beam, oriented to record the maximum tensile and compressive strains due to bending of the beam under load.

TABLE 2—*Frame load.*

Assembly No.	Duration of Test, min	δ (max), in.	Max Frame Load, lb	Avg Beam Temperature, deg F	Time, min
1.....	185	1.67	155 500	1002	170
3.....	215	3.32	144 800	1200	155
5.....	185	2.33	150 200	1032	170
8.....	90	2.26	98 000	1105	80
9.....	215	8.90	159 000	1150	150
10.....	160	15.0	177 000	950	145

FIG. 3—Stresses (based on recorded strains) developed during load application, in beams of assemblies 2, 5, and 9.

Method of Determining Stress in Supporting Beam

Strain gages were attached to the steel supporting members in each of the test assemblies described previously and located as illustrated by Fig. 1. The live load was applied in 10 per cent increments. Strain gage readings were recorded at each loading increment.

Method of Measuring End Thrust (Frame Load)

Strain gages were attached to the east and west sides of the restraining frame and located as illustrated by Fig. 2. The frames were calibrated by applying a known thrust between the two sides of the frame at the location of the support beams with the output of the strain gages recorded

at each increment of thrust. Strain readings were recorded for the duration of the fire tests of those assemblies designated as Nos. 1, 3, 5, 8, 9, and 10. The deflection and temperature of the support member were also recorded at significant intervals during the test.

Results

General

Information pertaining to the composite nature of the assemblies is contained in Table 1. Table 2 gives information on end thrust measured during the fire tests of Assemblies 1, 3, 5, 8, 9, and 10. Figure 3 shows the

FIG. 4—Simple composite beam.

stresses based on strains⁴ which were developed during load application in the beams of Assemblies 2, 5, and 9. The linear character of these curves indicates that at room temperature, elastic analysis may be applied to the structures under consideration. (Beam strains were not measured during the fire test.) Each of these structures displays a certain degree of composite action, as indicated by the difference in the stresses measured in the upper and lower flanges of the steel beam. As used in this paper, the term "composite action" means the load sharing of the support beam and the floor it supports by virtue of the shear connection existing between these two elements of the construction. Shear strength of $\frac{3}{4}$ -in. fusion (plug) welds used to secure steel deck section to supporting structural steel members varies with gage of the metal and is approximately 5000 lb for 16 gage deck.

⁴ An elastic modulus 30×10^6 psi was assumed.

Calculation of Degree of Composite Action

The following is concerned with the development of an expression defining the degree of composite action present in each of the tested assemblies.

The simple composite beam model shown in Fig. 4 may be employed for the purpose of analysis. The equivalent width of the slab is taken as $l/4$ in accordance with the design requirement of the current CSA Standard.⁵ The true effective width of concrete slab varies with the adequacy of the shear connection between the slab and the steel beam. The general calculation method is as follows (referring to Fig. 4):

$$Y = \frac{\frac{db}{n} \frac{d}{2} + A_s d_s}{A_t} \dots\dots\dots (1)$$

where

$$A_t = \frac{db}{n} + A_s \dots\dots\dots (1a)$$

Then the moment of inertia of the composite beam is:

$$I_t = \frac{bd^3}{12n} + \frac{bd}{n} \left(Y - \frac{d}{2} \right)^2 + I_s + A_s (d_s - Y)^2 \dots\dots\dots (2)$$

As is seen, this calculation procedure is based on transformed concrete area.

In attempting to analyze the actual beam-floor used in the fire test (which is not normally fully composite), elastic analysis may again be conveniently employed. The distance of the lower flange of the beam from the neutral axis ($\bar{h}_o$) may be determined from the measured stresses by means of the relation:

$$\bar{h}_o = \frac{\sigma_b D}{\sigma_b + \sigma_t} \dots\dots\dots (3)$$

The moment of inertia of the beam is then obtained from the expression

$$I = \frac{M \bar{h}_o}{\sigma_b} \dots\dots\dots (4)$$

which, combined with Eq 3 yields

$$I = \frac{MD}{\sigma_b + \sigma_t} \dots\dots\dots (5)$$

It should be noted that this method of analysis is based on the assump-

⁵ "Steel Structures for Buildings," *CSA Standard S16-1965*, Canadian Standards Assn., Ottawa, Ont., Canada.

FIG. 5a—Deflection curves.

tion that the neutral axis of the beam is located in the web of the steel beam; it is not valid for other cases. Finally, the degree of composite action (as calculated and entered in Table 1) is given by the expression:

$$C = \frac{I - I_o}{I_t - I_o} \dots \dots \dots (6)$$

in which

$$I_o = I_s + \frac{bd^3}{12n} \dots \dots \dots (7)$$

According to this definition for degree of composite action, $C = 0$ when $I = I_o$, and $C = 1$ when $I = I_t$.

Effect of Composite Action upon Structural Fire Endurance

From the practical point of view, the question arising from the results of the foregoing analysis is: To what degree does the composite nature

of an assembly influence its structural fire endurance? An examination of the deflection and rate of deflection⁶ curves illustrated in Figs. 5a and b, respectively, show that the simply supported noncomposite beam (No. 11) approached failure at an average temperature of 1185 F, a beam with a very low degree of composite action (No. 2) approached failure at an

FIG. 5b—Rate of deflection.

average temperature of 1270 F, and a fully composite beam (No. 7) had not approached failure at an average temperature of 1600 F.

⁶ In order that fire tests might be terminated prior to, but reasonably close to ultimate collapse, Robertson and Ryan ("Proposed Criteria for Defining Load Failure of Beams, Floors, and Roof Constructions During Fire Test," *Journal of Research*, National Bureau Standards, 63C, 1959, pp. 121–124) proposed that the point at which both $\delta = l^2/800 d$ and $\delta' = l^2/150 d$ can be regarded as an indication of load failure. In these expressions δ = maximum deflection, in., δ' = rate of deflection, in./hr, l = clear span of principal structural element, in., d = distance between the upper and lower extreme fibers of the principal structural element, in. These values have been marked by arrows in Fig. 6 to give an indication of the structural condition of the assembly at the time the test was terminated.

The fire test on the simple noncomposite beam, as well as more basic considerations,⁷ indicate that such members will become incapable of supporting their design loads at temperatures between 1100 and 1200 F. Review of the fire test results show that the degree of composite action existing in an assembly has the effect of prolonging the structural fire endurance period beyond that of a simple noncomposite beam, by an amount which is proportional to the degree of composite action.

Variation in the behavior of individual assemblies—(for example, in the case of Assemblies 3 and 9, the former showed no sign of rapid deflection at an average beam temperature of 1400 F, while the latter was approaching failure at the same temperature, although it had the greater

FIG. 6—End thrust.

degree of composite action) may be explained by a difference in the shear forces resulting from the applied load experienced during the fire test. (The live load on the beam in Assembly 9 was 1.73 times that on Assembly 3, although the strength of the welds was approximately the same in each case.) During the course of the fire test there is an audible indication of the failure of shear connections between the steel deck and the support beam. On assemblies where this has been observed, there is a marked increase in deflection rate after failure of the shear connections has been initiated. This indicates that significant reduction of the degree of composite action precipitates load failure for this type of assembly.

⁷ See p. 40.

Relationship of Applied Restraint and Degree of Composite Action and Their Combined Effect on Fire Endurance of an Assembly

Turning now to the question of restraint, Fig. 6 compares frame load (as obtained from the strain measurements) with average beam temperature for Assemblies 9 and 10 which incorporated steel beams of almost equal cross-sectional area. It is interesting to note that the maximum reaction was almost the same for both assemblies, but in the case of No. 9 the maximum point occurred at a much higher average beam temperature. This development is thought to result from the difference in degree of composite action inherent in these two assemblies. Tests at the Ohio State University⁸ indicate that there is an "optimum" degree of external restraint which can be applied. The fact that development of composite action between the beam and the floor reduces the magnitude of applied restraint at a given beam temperature may have the effect of optimizing the restraint of the steel beam.

The important practical effect of the applied or external restraint is to preserve the shear connection between the floor and the support beam—and, hence, the composite action. Therefore, the degree of composite action and applied restraint are factors which complement each other and, acting together, have the result of notably increasing the fire endurance of an assembly beyond that of a simple, noncomposite beam.

Conclusions

A review of the test data developed from the representative assemblies described in this paper indicates that the actual live load stresses developed in the steel support beams of such assemblies may be expected to be below the design working stresses to a degree which varies greatly from one assembly to another. In addition, the development of a varying degree of applied restraint against thermal expansion to both the steel beam and the concrete during the fire test has tended to obscure the significance of the structural behavior of these assemblies.

It is suggested that the existing standard might be modified to advantage by requiring more representative loading to assemblies proportioned by elastic design methods, thus re-establishing the intent of the original standard. With assemblies loaded in such a manner as to develop in each member the actual stresses contemplated by the design, there would be greater justification for a wider application of the data derived from such tests.

⁸ See p. 63.

DISCUSSION

*I. A. Benjamin*¹ (*written discussion*)—The authors are to be commended on having taken the time and effort over many years to compile actual stress and strain data on steel beams from test constructions.

The data which they have tabulated indicated quite clearly that over a fairly wide range of steel beam floor constructions, composite action is developed regardless of the theoretical stress calculations. Although the beams shown in Table 1 are presumed to have all been initially stressed to a calculated dead and live load stress of 20,000 psi, the highest stress measured in any floor construction consisting of steel deck and reinforced concrete was 15,600 psi or 78 per cent of the theoretical value.

The authors feel that the degree of composite action affects the critical temperature and rate of deflection of the beam, as they have shown graphically in Figs. 5*a* and *b*. They point out that the simply supported noncomposite beam No. 11 approached failure at an average temperature of 1130 F. However, this beam is not a "simply supported" beam in terms of structural design concept; but is a roller-supported beam—atypical for design considerations. The effective difference in restraint between roller supports and standard end connections has been markedly brought out in Professor Bletzacker's paper.²

As indicated in Professor Bletzacker's paper, roller-supported beams with no end restraint, having a range of measured fiber stresses from 16,900 to 22,000 psi, all failed within a few minutes of each other. This would indicate that the stress on the bottom fiber is not the only critical condition when the beam is on a theoretical roller-supported condition.

The authors' data would indicate that the stress in the extreme fiber does not tell the whole story, even for restrained beams. For example, beams No. 3 and 7 are both approximately 25 lb/ft sections. Although one has twice the stress in the bottom flange of the other and a range of composite action from 0.26 to 1.0, they exhibited similar performance.

Of considerable interest in the authors' paper is the observation that all the floor sections tested, composed of steel deck with concrete placed on top, showed average temperatures in excess of 1200 F before reaching limiting rates of deflection.

Although the data are not complete, we might infer from Figs. 5*a* and *b* that the critical factor is not the stress in the section but the amount of heat sink on top of the beam flange. The bar joist type of constructions, which are most sensitive to temperature, do not have the concrete slab

¹ Director of research, Granco Steel Products Co., St. Louis, Mo.

² See p. 63.

resting directly on the beam flange. The difference in performance between Test 2, the bar joist construction, and Tests 3, 7, and 9, which have the slab in contact with the top of the beam flange, could well be attributed to the difference in heat sink.

Figure 6 shows that the average end thrust occurs at a lower temperature for the bar joist than for the steel deck floor construction. This situation is consistent with the concept that a better heat sink will provide a greater differential between top and bottom flange, therefore allowing a higher temperature to occur in the steel before the maximum thrust is reached. The graph does not necessarily prove that the temperature difference at maximum restraint is the result of a variation in degree of composite action.

In summary, I agree with the authors' statement that the actual stresses developed in a typical floor construction are less than the theoretical design stresses—this is the nature of the design assumptions which are used. Any other loading requirement would be a forced condition and, particularly on certain types of reinforced concrete constructions, would require loading above design load to crack the concrete in order to develop the design stress in reinforcing bars.

The degree of restraint and the amount of heat sink directly in contact with the top flange of the beam may be the governing factors in beam performance.

N. S. Pearce and W. W. Stanzak (authors' closure)—The authors wish to thank Mr. Benjamin for his interest in their paper.

Mr. Benjamin has commented upon the data included in Figs. 5a and b which were included in the paper to illustrate the difference in performance exhibited by assemblies having varying degrees of composite action. From further review of the data, the authors would be most reluctant to conclude that the difference in the heat sink afforded by the assembly of bar joist construction, Assembly 2, contributed in a more significant way to performance of the assembly than did the difference in degree of composite action. The authors would refer to Harmathy's paper³ which clearly shows that the creep resistance of a beam at an average temperature of, say, 1600 F is so small that the beam would have difficulty in supporting its own weight. Therefore, in assemblies such as No. 7, as the result of composite action, it is conceivable that no appreciable bending stress was present in the support beam during the advanced stages of the fire test. As the average beam temperatures plotted in Figs. 5a and b included upper flange thermocouple readings, the heat sink effect does not necessarily explain the difference in structural performance of the various assemblies, although the relative coolness of upper beam flange and the floor deck is most necessary in preserving the composite nature of the assembly.

³ See p. 40.

With reference to Fig. 6, the authors agree that the greater temperature differential between the upper and lower portions of the beam might, in part, explain the temperature difference between the points of maximum thrust. The authors intend to pursue this point in their future investigations.

E. G. Butcher⁴ (written discussion)—The authors of this most interesting paper make the comment that the development of a varying degree of applied restraint against thermal expansion has tended to obscure the significance of the composite assemblies tested. The importance of this factor is now becoming generally realized and is probably one of the causes of variation in fire test results. The measured values of the end restraint which obtained in two of the tests reported in this paper indicate that these forces are considerable (up to 180,000 lb) (Fig. 6 and Table 2).

The authors ascribe the difference in the end thrust shown by the two curves of Fig. 6 as being due to the difference in degree of composite action inherent in the two assemblies. But is it not possible that small differences in the assembling into the test frame could have produced the same effect? The difference between the two curves in the 0 to 200 F region would seem to indicate that this might have been the case.

In discussing the effect of composite action upon the fire endurance, the authors make the point that a simply supported noncomposite beam approached failure at 1185 F (640 C), a beam with a small amount of composite action failed at 1270 F (680 C), but a fully composite beam had not approached failure at 1600 F (870 C). It is not clear, however, whether these results are due to any difference in the fire behavior of the composite beam or whether they are due to the fact that the loads used developed lower stresses in the composite beams than in the simple beam. Figure 3 and Table 1 would seem to indicate the latter.

N. S. Pearce and W. W. Stanzak (author's closure)—The authors also wish to thank Mr. Butcher for his interest. We would agree that a variation in the end thrust shown by the initial portion of the two curves of Fig. 6 could be produced by small differences in the method of assembly into the test frame. However, after reviewing the test data and recalling the care with which the structural steel beam supporting members were placed into the restraining frame, we would prefer to remain with the observations contained in the paper. As to the effect of composite action upon the fire endurance period, it was the intention of the authors to attribute the variation of fire endurance between assemblies incorporating support beams having no composite action, a small amount of composite action and fully composite action, to the variation in the initial stresses resulting from each assembly having been loaded as though the beams were simply supported.

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