Paper No. 6501

Steel frames with brickwork and concrete infilling†

by

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Discussion

Mr B. S. Smith (Lecturer in Civil Engineering, University of Bristol) wrote that Dr Holmes's suggested formula provided results in close agreement with his quoted experimental work. Whilst agreeing with him about the futility of an exact mathematical analysis of this problem, Mr Smith questioned the correctness of his simplified analysis when related to an actual structure.

19. In Fig. 1, which showed an infilled frame loaded laterally at the top corner by a force H, the Author applied a complementary vertical force $H \tan \alpha$ at the same corner to provide a diagonal resultant $H/\cos \alpha$ acting on the composite unit. He did not explain how $H \tan \alpha$ was provided in a structure but one could assume it mainly resulted from relieving the compressive load or even causing tension in the windward column, in which event the complementary vertical force should be applied to the lower corner of the frame on the windward side. For any tall building subject to extreme wind conditions the latter was quite possible. The effect on the infilling was as Dr Holmes assumed, a diagonal compressive load, but now there was the possibility of a tensile failure in the column or its connexions. Thus another mode of failure was apparent. Nor was this the only alternative mode, since tests by Benjamin and Williams[4] were reported to have frequently failed by a tension crack along the loaded diagonal and occasionally by the shearing of the foot of the lee column. Mr Smith had also recently carried out model tests on infilled frames and had obtained the first three types of failure.

20. Apart from these observed modes of failure, others could be anticipated, such as the buckling of the wall out of the frame.

21. Therefore Mr Smith suggested it was an oversimplification to consider only the problem of compressive failure of the infill when its occurrence was by no means certain.

22. However, to preclude other modes of failure and consider only the case of compressive failure of the infill, he agreed with Dr Holmes's basic approach. That was to assume that the infill behaved as a diagonal strut subsequent to partial separation from the frame.

23. Diagonal loading tests on a 12 in. × 12 in. × ½ in. mild steel frame with a ½ in. thick mortar infill, to represent approximately a 12 ft × 12 ft × 6 in. infill with an 8 in. × 6 in. × 35 lb rigid surround, showed separation over three-quarters of the length of each side. Light was visible through the gap while the unit was still elastic. This indicated that to consider the diagonal load distributed over the entire length of each side, even though assuming a triangular distribution of pressure, provided a calculated effective area of $dt/3$, to use Dr Holmes's parameter, which was excessive and incautious. To be conservative and more reasonable the area should be calculated assuming the load to be applied over a small area close to the corner. Several unframed mortar panels of sizes 6 in. × 6 in. × 1 in. to 12 in. × 6 in. × 1 in. loaded over a 2-in. length at diagonal corners had yielded effective areas varying from $dt/4$ to $dt/6$.

24. Dr Holmes had assumed that compressive failure would occur when the average

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diagonal strain reached a maximum value which had either been measured from cylinder tests or assumed. He had no doubt chosen to ignore the great variation of strain along the loaded diagonal in order to avoid complication. Measurements taken on the 6 in. × 6 in. × 1 in. to 12 in. × 6 in. × 1 in. mortar panels revealed that diagonal strains near the loaded corners were more than twice the average strain along the diagonal. These panels when loaded to destruction failed at average diagonal strains around 0·0015 whereas cylinders of the same mortar failed at strains around 0·0025. Since these compressive failures invariably occurred near the corners, it indicated that these increased local strains determined the strength of the panel.

25. This strain variation along the loaded diagonal was determined partly by the sides ratio of the panel and partly by the length of mutual contact of the frame and infill; the latter in turn being determined by the relative stiffness of frame and infill. The stiffer the frame, the greater length of mutual contact and the more even distribution of strain along the loaded diagonal, and therefore the greater the failing strength of the panels.

26. When considering Dr Holmes's formula for strength, a simple calculation revealed that for structures of practical proportions the frame would contribute only a small amount to the strength of the unit. Mr Smith suggested that a stiff frame's most important contribution to strength was in its better distribution of the load to the infill, resulting in reduced peak strains and the corresponding greater strength of the infill. In the post-ultimate range, after failure of the infill, the story was different and the frame would probably carry a large share of the load.

27. A further point on which Mr Smith commented was Dr Holmes's formula for the deflexion at failure or indeed at any other stage of loading up to failure. Despite all precautions he had found it impossible, even in laboratory conditions, to guarantee a tight fit of the infill in the frame. Field conditions would make a good fit practically impossible. Therefore deflexions in the initial stages of loading were a result of this slackness being taken up and would continue non-linearly until the frame bore fully on the infill. The unit would begin to act compositely and linearly. The load taken by the frame was approximately proportional to the total deflexion while that taken by the infill was proportional to the elastic deflexion. Because of the unpredictability of the fit any estimate of the total deflexion and of the load carried by the frame was likely to be grossly incorrect. However, since the frame would probably be carrying only a minor proportion of the load even when this extra deflexion due to slackness was considered, the resulting error in any strength calculation was unlikely to be important.

28. Dr Holmes's suggestions on this topic would be very welcome. However, Mr Smith felt that Dr Holmes had oversimplified this problem by ignoring other modes of failure which together with the mode he had considered, required considerable further investigation before any solution could be offered for use in the design office.

Mr R. J. Mainstone (Principal Scientific Officer, Building Research Station) wrote that Dr Holmes recognized the important contributions that infilling walls could make to the lateral strength and stiffness of the steel frames of tall buildings. The approximate method that he presented for estimating the strengths and stiffnesses of individual framed panels was simple and gave good predictions of ultimate loads in the cases considered. It was worth enquiring, therefore, whether the method was soundly enough based to be generally applicable in the design of tall buildings to resist sidesway.

30. Unfortunately, the range of the experimental data at present available was not wide enough to permit a final answer to this question. But much could be learnt from a fuller study of some of the data. What followed was based on such a study at the Building Research Station of the behaviour of the specimens numbered 3a, 3b, 6, and 7 in the Paper, in the light of tests on their component frames and walls acting separately and of more recent tests on small models. Specimens 3a, 3b, 6, and 7 were concrete-

encased steel frames with bolted joints and brickwork infilling, while the small models were welded steel frames with infillings of model brickwork.

31. It was clearly apparent from the tests, if not already intuitively obvious, that the behaviour of a steel frame with brickwork infilling was a composite one exhibiting a marked influence by each component of the composite structure on the behaviour of the other. By itself, the frame was capable of large side-sway accompanied by considerable plastic deformation while maintaining its strength, but it was never very stiff in resisting a sideways disturbing force. The wall, on the other hand, was very stiff until it cracked, in resisting such a force in its own plane, but it was capable, by itself, of only a very small movement before one or more diagonal cracks deprived it of most of its strength. In combination, the wall stiffened the frame, first by carrying most of the load itself, then, after cracking commenced, by changing the mode of deformation of the frame. And the frame, by holding the wall together after it cracked, prolonged the ability of the wall to share the load. The composite panel thus combined the initial stiffness and strength of the wall with the ductility and more slowly developed strength of the frame in a manner which Dr Holmes's equivalent structure (Figs 1 and 2 of the Paper) failed to represent.

32. Intuitively, it seemed likely that the actual mode of deformation and failure of the composite panel would depend both on the relative properties of the frame and the wall and on any other restraints on either—such as might be provided in a tall building by infillings adjacent to the panel considered. The maximum strength and stiffness were to be expected when the panel was so surrounded by other walls, the opposite extremes of behaviour in the absence of surrounding walls being represented by the combination of a relatively weak frame with a strong infilling and of a relatively strong frame with a weak infilling.

33. Specimens 3a, 3b, 6, and 7 were all single panels without surrounding walls. But they did cover an interesting range of relative strengths and stiffnesses. Their behaviour was analyzed in Figs 3–5 and in Tables 2–4. The theoretical loads in the latter were calculated from Dr Holmes's formulae, the values for the uncased frame and wall differing slightly from those in Table 1 of the Paper, presumably because of a

TABLE 2.—SPECIMENS 3A AND 3B (MEANS). GIRDERS 10 IN. × 4½ IN. (ENCASED 12 IN. × 8 IN). STANCHIONS 10 IN. × 8 IN. (WEAK WAY) ENCASED 12 IN. × 12 IN. WALL 4½ IN. BRICK

	Experimental load P tons taken by:			Theoretical load $P(=H)$ tons taken by:		
	Frame	Wall	Frame + wall	Frame	Wall	Frame + wall
Encased frame and wall separately . .	20	20	40			
Bare frame + wall at $S_H = 0{\cdot}72$ in.* . .				8·1	39·8	47·9
Encased frame + wall at $S_H = 0{\cdot}72$ in.* . .	26	23	49	23·4	39·8	63·2
Encased frame + wall at maximum . .	30	22	52			

* S_H is calculated on the frame centre-line dimensions and corresponds to $x = 1{\cdot}22$ in. (horizontal deflexion as measured). Wall strengths are calculated on actual wall dimensions.

TABLE 3.—SPECIMEN 6. FRAME AS IN SPECIMENS 3A AND 3B. WALL 13½ IN. BRICK

	Experimental load P tons taken by:			Theoretical load $P(=H)$ tons taken by:		
	Frame	Wall	Frame + wall	Frame	Wall	Frame + wall
Encased frame and wall separately . .	20	60	80			
Bare frame + wall at $S_H=0{\cdot}72$ in.* . .				8·1	119·6	127·7
Encased frame + wall at $S_H=0{\cdot}72$ in.* . .	38	74	112	23·4	119·6	143·0
Encased frame + wall at maximum . .	24	111	135			

* S_H is calculated on the frame centre-line dimensions and corresponds to $x=1{\cdot}22$ in. (horizontal deflexion as measured). Wall strengths are calculated on actual wall dimensions.

TABLE 4.—SPECIMEN 7. GIRDERS 13 IN. × 5 IN. (ENCASED 17 IN. × 8 IN.). STANCHIONS 10 IN. × 8 IN. (STRONG WAY) ENCASED 14 IN. × 12 IN. WALL 4½ IN. BRICK

	Experimental load P tons taken by:			Theoretical load $P(=H)$ tons taken by:		
	Frame	Wall	Frame + wall	Frame	Wall	Frame + wall
Encased frame and wall separately . .	23	20	43			
Bare frame + wall at $S_H=0{\cdot}72$ in.* . .				29·5	39·8	69·3
Encased frame + wall at $S_H=0{\cdot}72$ in.* . .	19	55	74	57·6	39·8	97·4
Encased frame + wall at maximum . .	20–23	55–52	75			

* S_H is calculated on the frame centre-line dimensions and corresponds to $x=1{\cdot}22$ in. (horizontal deflexion as measured). Wall strengths are calculated on actual wall dimensions.

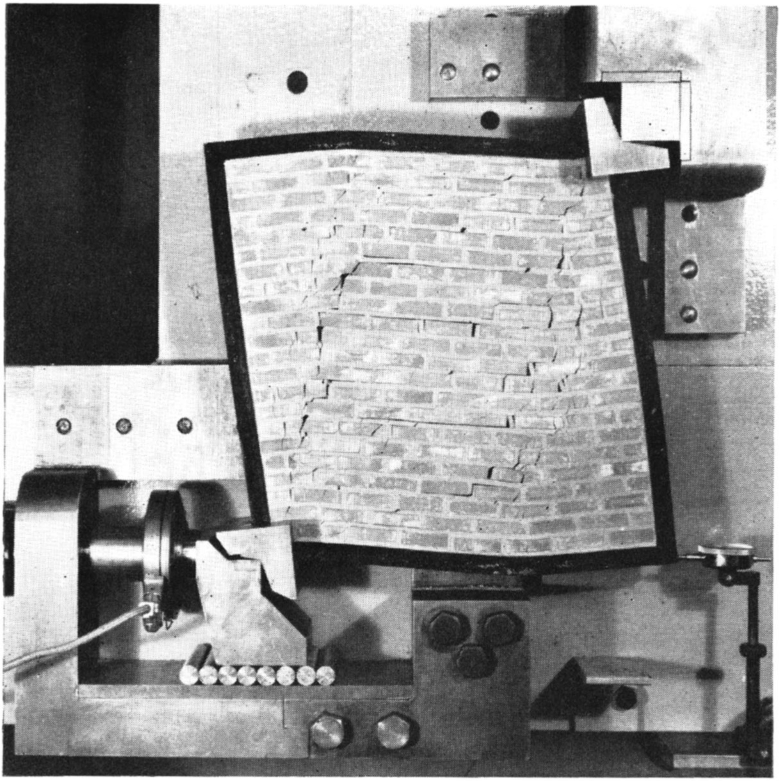

FIG. 6.—A MODEL FRAME WITH MODEL BRICKWORK INFILLING, SHOWING THE CHANGED MODE OF DEFORMATION OF THE FRAME DUE TO THE PRESENCE OF THE WALL. THE PANEL STILL CARRIED MORE THAN 90% OF ITS PEAK LOAD AT THIS LARGE DEFLEXION

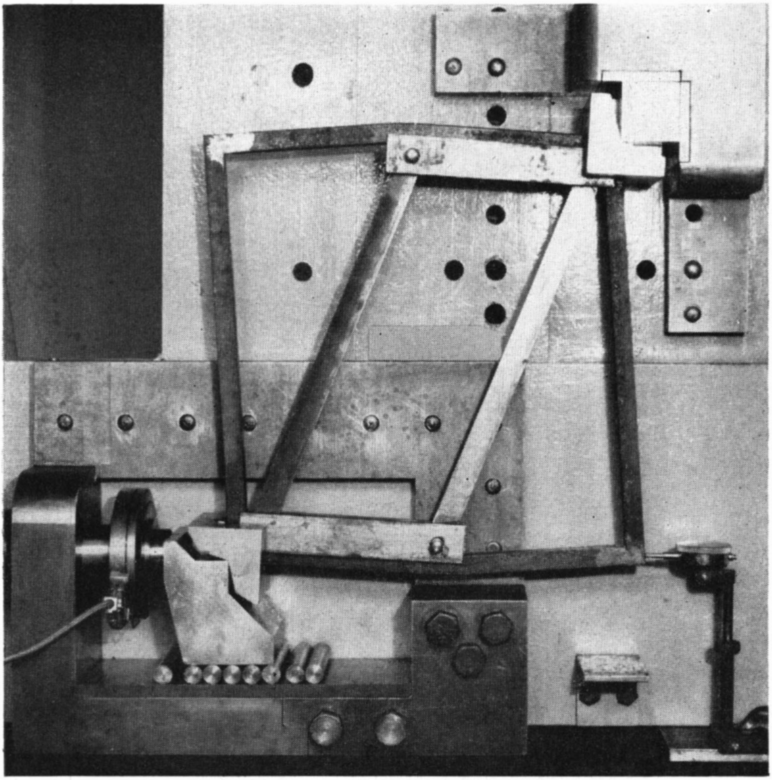

FIG. 7.—A MODEL IN WHICH THE WALL IN FIG. 6 IS REPLACED BY A PINNED LINKAGE THAT MODIFIES THE MODE OF DEFORMATION OF THE FRAME IN A SIMILAR MANNER. IN THIS MODEL THE WHOLE EXTERNAL LOAD IS RESISTED BY THE FRAME IN BENDING. IF THE HORIZONTAL MEMBERS OF THE LINKAGE WERE MADE MORE NUMEROUS AND CONTIGUOUS, THE CONTRIBUTION OF THE CRACKED WALL TO THE STRENGTH OF THE PANEL WOULD BE ANALOGOUS TO FRICTION BETWEEN THESE HORIZONTAL MEMBERS

different interpretation of the lengths l and h. Of the experimental loads, only those for the frame and wall acting separately and for the frame + wall acting compositely were based directly on observations. The curves in Figs 3 and 4 for the frame acting compositely were derived from those for the frame acting alone by making appropriate corrections for the observed differences in the modes of deformation and for the greater extent of cracking in the encasement that resulted from these changes of mode. No such curve was given in Fig. 5 because the change of mode was not sufficiently well defined in specimen 7: there was here almost certainly some gain in frame strength when acting compositely but its amount could not be confidently assessed. The curves for the wall acting compositely were, finally, obtained by difference.

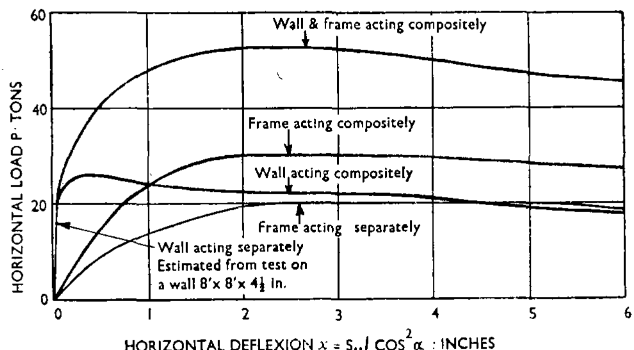

FIG. 3.—MEAN LOAD/DEFLEXION CURVES FOR SPECIMENS 3A AND 3B

34. The load/deflexion curves called for the following comments:

(*a*) The initial behaviour of each composite panel was largely determined by the wall. The unframed wall, on which the estimates for the walls acting separately were based, failed by cracking diagonally at a compressive strain, e'_c, of the order of only 0·0001, and the framed walls cracked at roughly the same strain. Before this cracking, the whole structure was elastic and very stiff.

(*b*) After this cracking, the stiffness of the walls, and hence of the whole panels, decreased. The walls continued to carry load, though initially still mainly by diagonal compression, but also, to an increasing extent as deformation increased, by internal friction at the cracks. Indeed the load carried by the walls increased for a time, the stronger and stiffer frame of specimen 7 leading to a proportionately greater increase than the weaker frame of the other specimens. Qualitatively, it could be seen that the effective "ductility" of the wall due to the binding action of the enclosing frame was least with the combination of weak frame and strong wall in specimen 6. The behaviour of this wall came closest to that of the wall on its own, but it was still very different. The effective "ductility" of the infilling wall was obviously of great importance when designing either for strength or for energy absorption.

(*c*) The wall not only contributed its own strength and stiffness to the composite panel but also stiffened the frame initially and increased both its elastic and plastic strengths. The changed mode of deformation of the frame which brought this about was seen clearly in Fig. 23b of Dr Wood's Paper[1] and in Fig. 6. It could be simulated by replacing the wall by a linkage as in

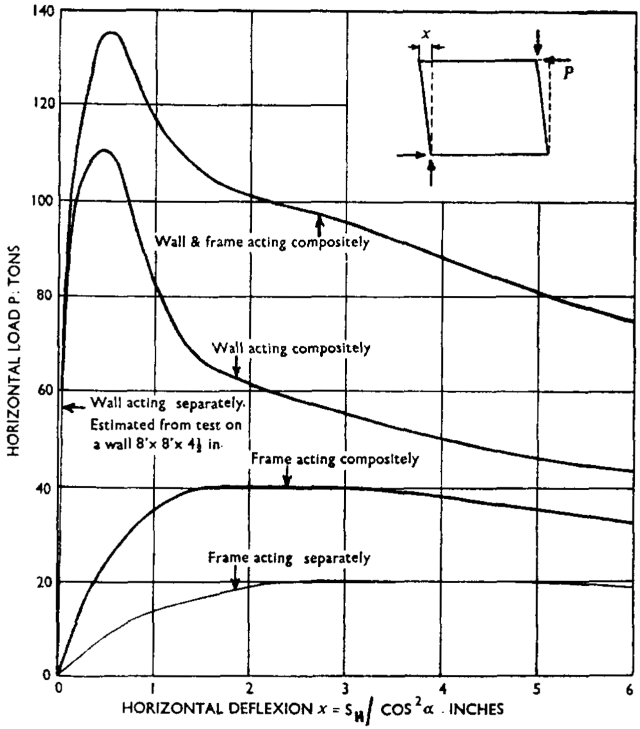

FIG. 4.—LOAD/DEFLEXION CURVES FOR SPECIMEN 6

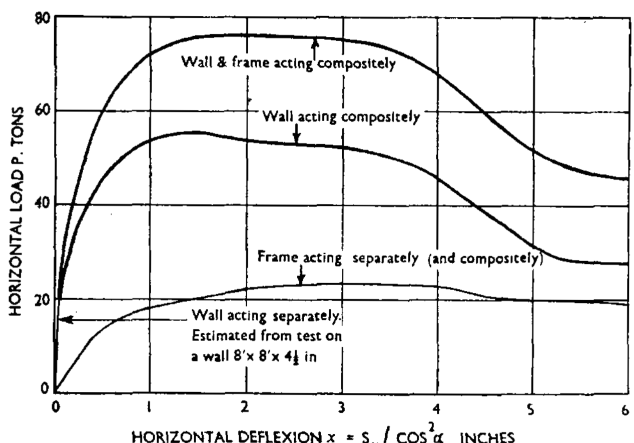

FIG. 5.—LOAD/DEFLEXION CURVES FOR SPECIMEN 7

Fig. 7.* The stronger the wall in relation to the frame, the further along the beams did the hinges move and the greater was the increase in the load carried by the frame. The loss in strength of each frame at fairly large deflexions was due only, in the range covered by the curves, to progressive cracking of the concrete encasement at large hinge rotations. The model frames, without encasement, had shown no such loss and, in consequence, their infilling walls had shown little loss of strength even at very large deflexions—one wall actually having gained in strength. The rather anomalous behaviour of the frame of specimen 7 was to be explained in part by the weakness of its joints in relation to the frame members. This specimen suggested, perhaps, the possible behaviour of a panel having a rather weaker frame but stiffened by surrounding walls.

35. Comparison of the experimental and theoretical strengths and deflexions showed that:

(1) The calculated values of S_H were quite unrepresentative of the initial stiffnesses of the composite panels. They were sometimes fortuitously close to the deflexions at which the maximum loads were reached, but, for a brick infilling, they seemed to have little real physical significance.
(2) The total theoretical strengths were of about the right magnitude, but the correlations between the theoretical and experimental values of the contributions of the frames and walls to these strengths were less good. Whether the errors in the latter would always compensate for one another could only be decided on the basis of further experimental data covering a wider range of frame and wall strengths. On present evidence, Mr Mainstone,[5], like Dr Wood[6], felt that some simple interaction formulae would better represent the composite nature of the total strength and would be more likely, therefore, to be generally applicable.

36. Further data were urgently needed and any tests, such as those made by Dr Holmes, were most welcome. They would be doubly welcome if, when published, the ultimate loads were supplemented by full details of the specimens and by load/deflexion diagrams.

Dr R. H. Wood (Senior Principal Scientific Officer, Building Research Station) wrote that Dr Holmes's paper was a welcome contribution to the theory of composite action applied to the wracking strength of framed panels. It showed clearly that, although composite action was an involved subject, (meaning in the limit the behaviour of the whole structure), it was quite feasible to produce simple design rules which, when coupled with a suitable load factor, could be conservative. Indeed, one of the most important findings of such studies of composite action was that it was often better to have a simplified theory for the whole structure than an "exact" theory for an idealized part of it, which exact theory could not operate without assuming the loads on the idealized part.

38. Whilst Dr Wood sympathized with Dr Holmes's approach, the choice of the crushed diagonal band of brickwork as the next most important consideration after the bent surrounding frame was not perhaps the most powerful choice. It was admittedly the one which first came to the mind but it was nevertheless one which Dr Wood had abandoned several years ago, since the diagonal strength of the band of crushed brickwork was nearly unlimited, the width of the band not being defined *a priori*. The use by the Author of the limiting compressive strain (following Prof. A. L. L. Baker's suggestions of values between 0·002 and 0·005) was an ingenious device which provided extra information to link together the frame action and the diagonal action, but the figure of 450 lb/sq in. was very low for Fletton brickwork at its ultimate strength. Moreover it did not overcome the difficulty of stating the equivalent width of the diagonal band, for

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* Figs 6 and 7 are photographs and are placed between pp. 96–97.

which the Author had proposed instead a linearly increasing boundary reaction between beam and wall.

39. Now such a linear beam reaction arose from the Author's sketch (Fig. 1) of the failure mechanism, which was somewhat at variance with the form of failure shown in the photograph of a panel at collapse in Dr Wood's earlier Paper (Fig. 23b of Reference 1). The plastic hinge which developed at about the third point of the span clearly demonstrated considerable local thrust, associated with jamming of the brickwork in the corner where the Author had depicted separation. (This was strongly reminiscent of the jamming action particularly found in the corners of continuous floor slabs supported on beams, which gave rise to intensive membrane action[7]). This jamming action must be a function of the stiffness of the beams, and in turn it would raise the shear strength of the brickwork. The main point however was that the direct compression along the diagonal was partly destroyed by the shear cracking, and composite action took place more by arching and jamming across the unloaded corners.

40. A way of short-circuiting a mass of confusing detail was to obtain separate indications of the collapse loads according to idealized theories and then to state the actual collapse load as an "interaction formula". An example of such a treatment was the now well-known Merchant-Rankine formula for collapse of multi-storey frames[1]. Whereas the Author coupled the limiting "plastic" strain of the brickwork on the diagonal with an *elastic* analysis of the framework, Dr Wood had alternatively given[8] a possible interaction formula in terms of the two separate ultimate strengths of the wall and frame which were easy to determine. Whereas such an interaction formula required some improvement to account for the beam-stiffness effect mentioned above, it brought out the point that one must *invent* a sliding scale of load factors (λ) for the transition range between brittle failure of the wall only (say $\lambda=8$) to plastic failure of the frame only (say $\lambda=2$). Such transition load factors supported the Author's contention that simple formulae were not only feasible but imperative.

41. Whilst simplified formulae were in the process of being developed, every author should make an attempt to present the load/deflexion curves for any test results quoted. The question was often asked whether the collapse loads of several panels could be added together when the panels were in the same storey of a building. If the load/deflexion curves showed sufficient ductility, this simple addition became feasible, or in the event of insufficient ductility, reduction could be forecast. For the benefit of the reader, some of the load/deflexion characteristics of the panels quoted in reference (1) were given in references (9) and (10) below.

Mr Salio Sachanski (Scientific Assistant, Building Research Institute, Sofia, Bulgaria) wrote that the Author's formulae for the calculation of the force at failure and the deflexion of the brickwork and concrete infilling, encased in a steel frame, were rather approximate and in some cases led to considerable deviations from the actual values.

Notation

43. Mr Sachanski's notation was:

$h, l, d, t,$ denote height, width, diagonal length, and thickness of wall infilling,
$H,$ denotes horizontal shear force at failure,
$R_s,$ denotes shear strength of infilling,
$f,$ denotes coefficient of internal friction,
$f=0{\cdot}7$ for compact bricks,
$f=0$ for lattice bricks,
$\beta=l/h,$
$\xi_2,$ denotes compressive strain of diagonal,
$f_c,$ denotes compressive strength of infilling in the direction of the frame diagonal.

44. Investigations had been carried out in the Building Research Institute in Sofia to establish the carrying capacity and deformation of brickwork and concrete infilling[12] encased in reinforced concrete frames and loaded with horizontal forces. The bond

between infilling and frame was good, and therefore the two elements worked jointly without cracks between them. Models without bond between infilling and frame were tested too, the number of these tests being smaller.

45. Investigations for the determination of the carrying capacity and deformation of brickwork encased in a steel frame had been carried out in the Soviet Union by S. V. Poliakov[11]. In these tests the steel frame had joints at the corners, and therefore took no part in carrying the load, but only transferred it to the brickwork.

46. The law which governed the transfer of the external force H to the brickwork was based on experimental data. The stresses in the brickwork itself were determined from the assumed loading; and were subsequently used for determining the horizontal force at failure:

$$H = 0{\cdot}8\ lt\ R_s \frac{1-0{\cdot}1\beta}{0{\cdot}75\ f/\beta} \qquad (5)$$

47. The compressive strain of the diagonal on the appearance of the first crack along the diagonal was determined from

$$\xi_2 = \left(\frac{2H}{lt}\right)^2 \cdot D \qquad (6)$$

where

$$D = 11{\cdot}9.\ 10^{-6}[\beta(\beta-2)+2{\cdot}93] \qquad (7)$$

48. The horizontal force at failure according to the Author was determined by

$$H = F\cos\alpha = \frac{d}{3}\, tf_c \cos\alpha \qquad (8)$$

49. The compressive strain of the diagonal was assumed to be

$$\xi_2 = 0{\cdot}005. \qquad (9)$$

50. Comparing these two methods, using only some of Poliakov's test results, (Tables 5 and 6) since they were carried out under similar conditions, the following conclusions might be drawn:

(*a*) The carrying capacity of the brickwork determined from equation (1) did not differ much from the test results. Taking into consideration that the investigations were carried out upon 64 models with or without openings, it might be assumed that they gave a true impression of the behaviour of the brickwork encased in the steel frame. The carrying capacity depended on the bond between bricks and mortar, which could easily be determined experimentally or from the literature[11].

(*b*) The infilling was in a complicated state of stress. There existed compressive, shearing and tensile stresses[11,12]; therefore Dr Holmes's compressive strength, f_c, of the infilling in the direction of the frame diagonal, was conditional. Indeed f_c depended on the mortar and the angle α but there were no indications for its determination. Neither was a method given for the determination of f_c. On account of this the carrying capacity at the cited tests could not be obtained from eq. (8). The Building Research Institute, Sofia considered it incorrect to determine f_c by means of

$$f_c = \frac{H}{d/3.t\cos\alpha} \qquad (10)$$

since this would require a great number of tests under different conditions. The establishment of a composite strength, f_c, impeded the investigations and might lead to serious errors for brickwork which had not been tested.

(*c*) Shearing and tensile stresses appeared in the infilling. The force at failure in the brickwork should be determined from a "simple" strength (shearing—according to Poliakov; compressive perpendicular to the joints; or tensile).

(*d*) The width $d/3$ of the equivalent strut was determined rather approximately. It depended on the relation between the stiffnesses of the frame and of the infilling. This was also confirmed by photoelastic tests at Sofia which although carried out with an isotropic material, gave an approximate notion of the stresses in the infilling and the frame.

(*e*) Table 6 showed the strains obtained by R. H. Wood[13] on the appearance of the first visible crack. Dr Holmes gave Wood's results but with deflexions arising at failure.

(*f*) In Bulgarian building practice, the ultimate deflexion of the brickwork under

TABLE 5.—RESULTS OF POLIAKOV'S TESTS OF INFILLING

Specimen No.	Compressive strength		Bond of mortar	Dimensions of brickwork: cm			Horizontal force at failure: tons		
	of bricks	of mortar		l	h	t	experimental	according to (5) of Poliakov	according to (8) at $f_c = 31{\cdot}5$
	kg/cm²	kg/cm²	kg/cm²	cm	cm	cm			
6, 7	75	103	6·4	120	120	25	15	14·2	31·6
11	75	93	5·5	120	120	25	13	13·4	31·6
18	100	106	3·1	120	120	25	19	14·4	31·6
35	75	50	3·2	300	200	25	17	16·6	79
36	150	90	4·8	300	200	25	17	23·8	79
46	75	66	2·5	300	200	25	34	30·8	79

TABLE 6.—SPECIFIC DEFLEXION IN DIRECTION OF FRAME DIAGONAL ON APPEARANCE OF DIAGONAL CRACK ACCORDING TO POLIAKOV'S AND WOOD'S RESULTS

Specimen No.		Dimensions of brickwork: cm			Strain	
Wood*	Poliakov	l	h	t	experimental	theoretical
	6	120	120	25	0·0014	0·002
	11	120	120	25	0·0015	0·0018
	18	120	120	25	0·0025	0·0020
	35	300	200	25	0·0005	0·0003
	36	300	200	25	0·0003	0·0005
	46	300	200	25	0·0016	0·0009
3a		311	280	11·5	0·0023	
4		311	280	11·5	0·0020	
5a		311	280	7·6	0·0019	
5b		311	280	7·6	0·0021	
5c		311	280	7·6	0·0031	
6		311	280	34	0·0019	
7		311	280	11·5	0·0021	

* The wall-panel numbers are those used by Dr Holmes.

horizontal force was considered to be the one at which the first diagonal crack appeared in the brickwork and not the deflexion at which the loading reached its maximum value. The object was to safeguard the brickwork against failure due to its low deformative capacity. The first diagonal crack occurred at rather small deflexions[11,12], after which these quickly increased with increasing load. To assume that the deflexion at collapse of the brickwork was the ultimate one would mean that the brickwork was no longer fit to carry load, that it had been seriously damaged and could no longer fulfil its purpose.

(*g*) Poliakov's strains had a certain value for bricks of dimensions 120×120 mm and another for bricks of dimensions 300×200 mm (Table 6). Therefore the ultimate deflexions of the brickwork depend on the relation $\beta = l/h$. This effect was determined from equation (7).

(*h*) The strains, at which the first diagonal crack appeared, were according to Wood about 0·002. They had been determined for $\beta = l/h \approx 1$ and coincided with Poliakov's results for $\beta = 120/120 = 1$. In another ratio of length to height the ultimate deflexion would be different. This was evident from Poliakov's results and from equation (7).

51. The Author's formula for the determination of the force at failure in the infilling was simple, but its application was impeded by the difficulty of determining, f_c, for which no indications were given. The formula suggested by Poliakov was more complicated but easier to use since the shear strength R_s could be determined experimentally for all bricks and mortars.

52. It was more correct to regard the deflexion on the appearance of the first diagonal fissure, $\xi_2 = 0{\cdot}002$, as the ultimate deflexion at horizontal loading, and not the deflexion at failure $\xi_2 = 0{\cdot}005$, when the infilling could no longer fulfil its function.

53. The adoption of an ultimate deflexion for all sorts of infilling regardless of the dimensions of the bricks and the mortar was not quite exact; therefore the adoption of a certain ultimate deflexion should be considered as rather approximate.

The Author, in reply, said that the behaviour of steel frames with infilling walls was undoubtedly very complex, and any attempt at analysis must make simplifying assumptions. The contributions to the discussion were mainly concerned with the degree of approximation involved in the assumptions made in the Paper and the extent to which the proposed analysis could be used in design.

55. The Author was in agreement with Mr Smith when he said that for structures of practical proportions the frame would contribute only a small amount to the strength of the unit as a whole, and that the main contribution of the frame to strength was in ensuring a favourable distribution of load to the infill. The approach suggested by Dr Wood and Mr Mainstone involved an assessment of the separate strengths of infill and frame subsequently combined in an "interaction formula" to give the composite strength. Such an approach would have to allow for the favourable way in which the load was distributed to the infill by the frame; such an allowance would be essentially very similar to the Author's concept of an effective width of infill.

56. It was true, as Mr Smith stated, that it was impossible to guarantee a tight fit of the infill in the frame even under laboratory conditions, any slackness of fit would cause larger deflexions to occur, but would not affect the load-carrying capacity too seriously, in fact, slackness of fit probably went a long way towards accounting for the variation in strength of the nominally identical specimens referred to in the Paper. Because of this unpredictability of the "fit" of the infill in the frame, very large errors could arise in calculations of the type which Mr Mainstone presented in Figs 3–5 and Tables 2–4 where the separate strengths of the frame and infill were deduced from observed deflexions. Thus equation (3) should be used to evaluate the maximum load and not, as shown in Tables 2–4, the load at the experimental ultimate-load deflexion. (Because the load/deflexion curves tended to the horizontal at ultimate load there was

additional error introduced into Tables 2–4 as the experimental ultimate-load deflexion was somewhat indeterminate).

57. Because of the possibility of lack of fit of the infill in the frame, the formula for the ultimate-load deflexion given in the Paper should only be regarded as giving the "order of deflexion", however the effect of lack of fit on strength was much less pronounced. In a typical case an infill could increase the strength of a frame by 400%, so that this could be allowed for by a safety factor, still leaving a substantial gain.

58, Both Dr Wood and Mr Sachanski referred to the value of 450 lb/sq. in. used for f_c with brickwork. Where the infilling was monolithic, crushing took place at failure on the compression diagonal. If the infilling consisted of small units with mortar joints, failure occurred by movement at the joints before crushing on the compression diagonal occurred. The stress at which this movement occurred must depend on friction at the joints and the size of the units used for infilling. The value of 450 lb/sq. in. was based on the test results analysed as stated in the Paper.

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