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Structural Experiments on a Building of Composite Design

Essais pratiques sur un ouvrage de conception mixte

Versuche am Tragwerk eines Gebäudes in Verbundbauweise

S. R. SPARKES J. C. CHAPMAN A. C. CASSELL

Introduction

The major expansion of Imperial College, London, involves the construction of a number of new buildings. Where it seems appropriate the opportunity is being taken to carry out structural experiments on the buildings. Some short-term tests on a minor building have already been described [1], and the present paper describes experiments on part of the new building for Mechanical Engineering.

The building is of rather unusual construction, having castellated steel beams which are designed to act compositely with the floor slabs. The first stage of the building (figs. 1 and 2) will be occupied towards the end of 1959, and consists of a main spine having nine floors, with two six-storey spurs joined by a lower laboratory block. The spine has a reinforced concrete core containing the lifts and staircases, and a single bay steel framework with simply supported composite beams; the laboratory link is of steel portal construction. To ensure composite action between beam and slab, welded stud shear connectors are provided.

In the spine the beam-column connexions were made by clearance bolts to bottom brackets and web cleats. In the portals, site welded joints were made in the beams at about one fifth of their span from the columns. Column splices were site welded.

The investigation was planned to give information on the stresses in selected beams and columns due to erection of the steelwork, during and after the casting of the concrete slabs and encasement, and subsequently during the life of the building. This paper includes measurements made almost

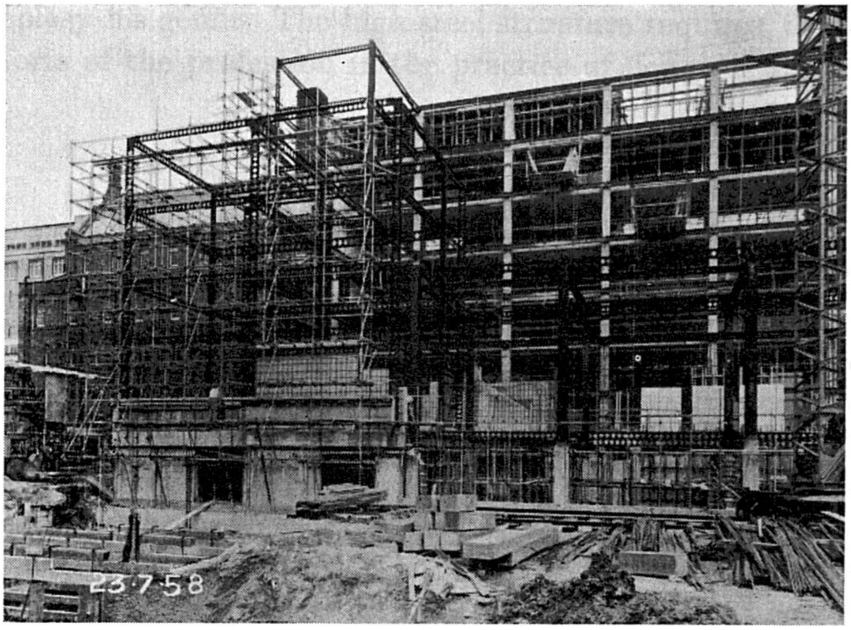


Fig. 1. Mechanical Engineering. Building Stage 1. Imperial College.

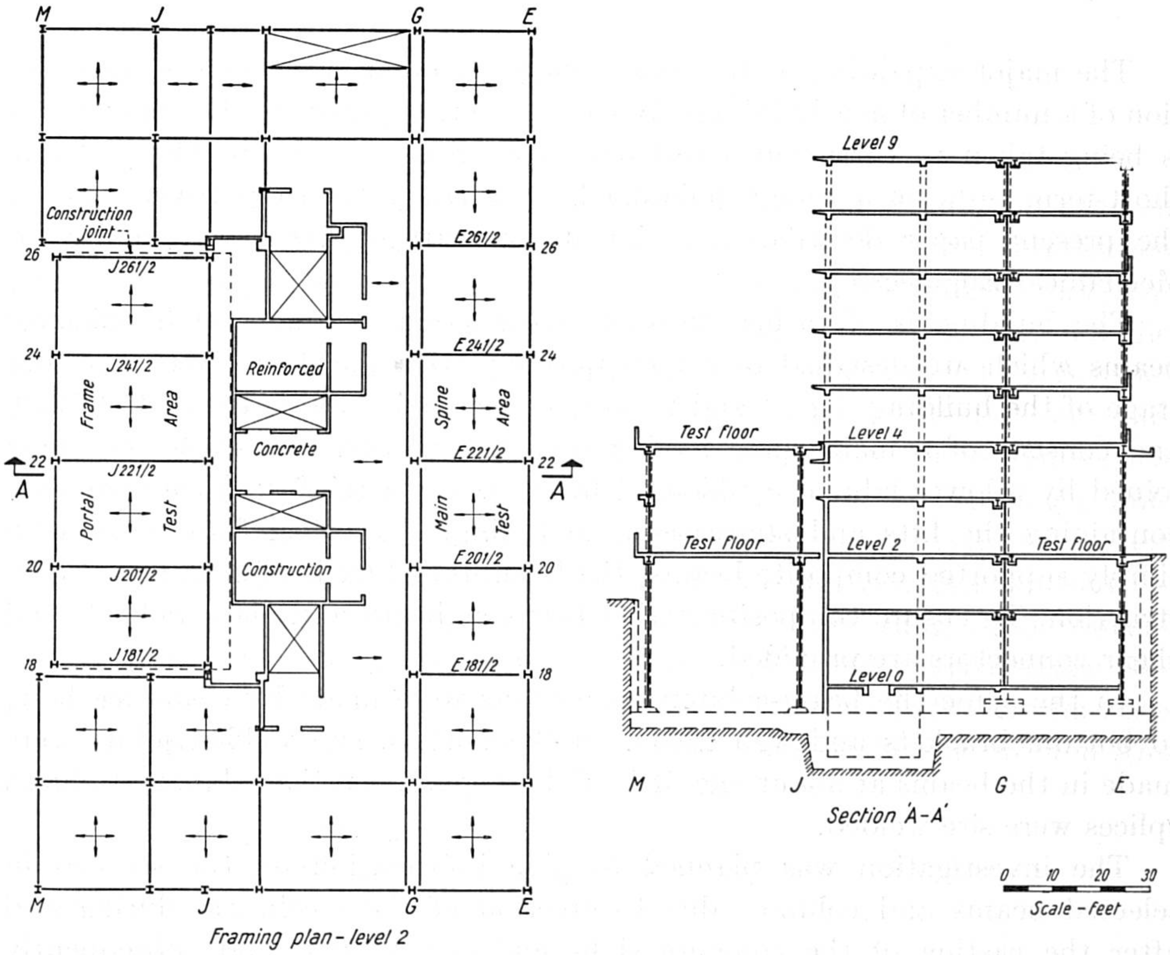


Fig. 2. Mechanical Engineering Building Stage 1. Imperial College.

up to completion of the building. Short-term live loading tests were also made to study beam and slab interaction, and since these tests were necessarily non-destructive, parallel laboratory tests to failure are proceeding, but these are not reported in the present paper.

Instrumentation and Live Loading

Strain gauges were required to be of robust construction and to have long term stability, and for these reasons vibrating wire gauges were chosen. The gauge [2] is shown in fig. 3, and incorporates welded studs for fixing the taut wire, the electro magnet, and the cover. The gauges were fixed to the steel members and sealed in the laboratory before erection, and wire frequencies were recorded with the members in a stressfree condition; this measurement provided the datum to which later strain measurements were referred. Fig. 4 shows gauges mounted on the flanges of a beam, and the disposition of the gauges is shown in fig. 5.

Deflexions of beams and slabs under live loading were measured by dial gauges relative to tensioned wires spanning between columns, contact with the wire being detected electrically. Any settlement of the columns will be measured

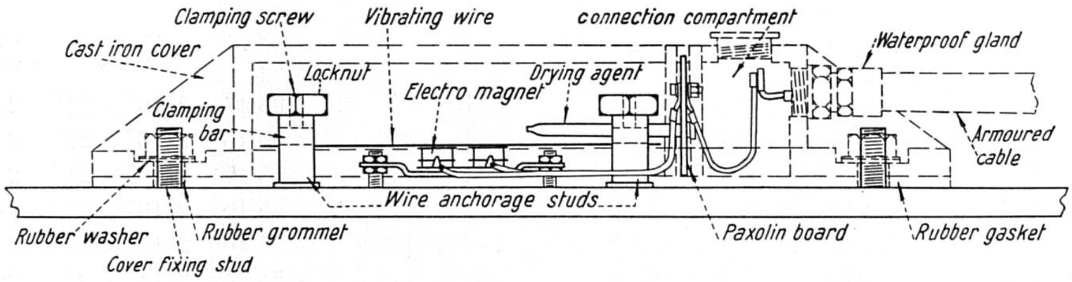


Fig. 3. Strain Gauge Assembly.

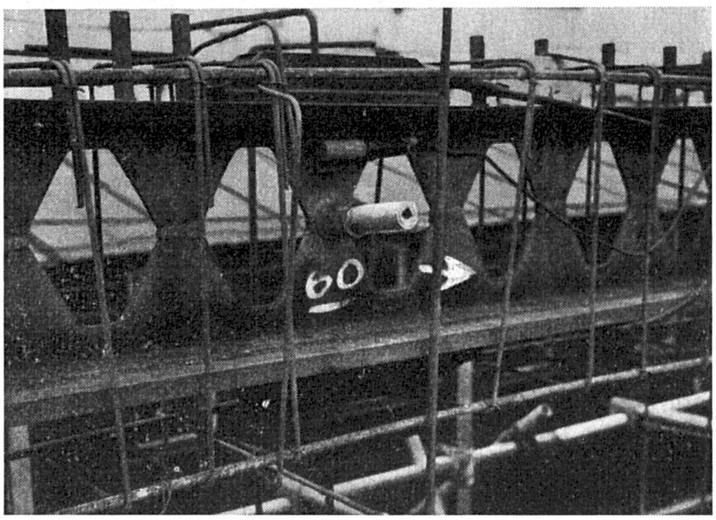
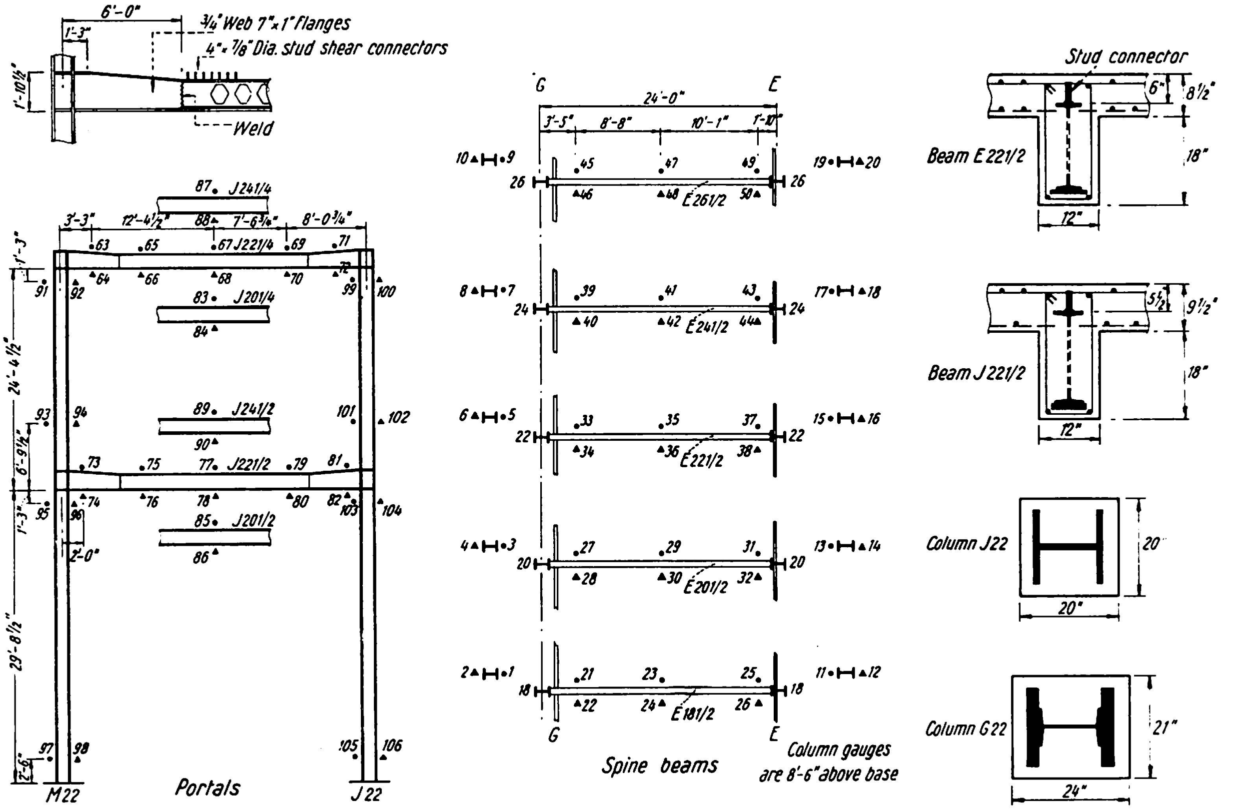


Fig. 4. Castellated Beam with Shear Connectors, Strain Gauges and Resistance Thermometer.



Beam	Section	Flange Plate	Depth below concrete
E 181/2	$18'' \times 5'' \times 32$ lb C.	$1/7'' \times \frac{3}{4}''$	$5\frac{1}{2}''$
E 201/2	$18'' \times 5'' \times 32$ lb C.	$1/8'' \times \frac{3}{8}''$	$6''$
E 221/2	$18'' \times 5'' \times 32$ lb C.	$1/8'' \times \frac{1}{2}''$	$6''$
E 241/2	$18'' \times 5'' \times 32$ lb C.	$1/8'' \times \frac{5}{8}''$	$6''$
E 261/2	$18'' \times 5'' \times 32$ lb C.	$1/8'' \times \frac{5}{8}''$	$6''$
G 200/2	$12'' \times 6'' \times 44$ lb	$2/8'' \times \frac{3}{4}''$	$2''$
E 200/2	$12'' \times 4'' \times 18$ lb C.	$1/8'' \times 1''$	$6''$
J 201/4	$18'' \times 5'' \times 32$ lb C.	$1/8'' \times \frac{7}{8}''$	$6''$
J 221/4	$18'' \times 5'' \times 32$ lb C.	$1/8'' \times \frac{7}{8}''$	$6''$
J 241/4	$18'' \times 5'' \times 32$ lb C.	$1/8'' \times \frac{3}{4}''$	$6''$
M200/4	$12'' \times 4'' \times 18$ lb C.	$1/8'' \times \frac{3}{4}''$	$6''$
M220/4	$12'' \times 4'' \times 18$ lb C.	$1/8'' \times \frac{3}{4}''$	$6''$
J 200/4	$12'' \times 4'' \times 18$ lb C.	$1/8'' \times \frac{7}{8}''$	$6''$
J 220/4	$12'' \times 4'' \times 18$ lb C.	$1/8'' \times \frac{7}{8}''$	$6''$
J 201/2	$18'' \times 6'' \times 44$ lb C.	$1/7'' \times 1''$	$5\frac{1}{2}''$
J 221/2	$18'' \times 6'' \times 44$ lb C.	$1/7'' \times 1''$	$5\frac{1}{2}''$
J 241/2	$18'' \times 6'' \times 44$ lb C.	$1/7'' \times 1''$	$5\frac{1}{2}''$
M200/2	$15'' \times 4\frac{1}{2}'' \times 25$ lb C.	$1/7'' \times \frac{5}{8}''$	$5\frac{1}{2}''$
M220/2	$15'' \times 4\frac{1}{2}'' \times 25$ lb C.	$1/7'' \times \frac{5}{8}''$	$5\frac{1}{2}''$
J 200/2	$15'' \times 6'' \times 40$ lb C.	$1/7'' \times 1''$	$5\frac{1}{2}''$
J 220/2	$15'' \times 6'' \times 40$ lb C.	$1/7'' \times 1''$	$5\frac{1}{2}''$

Fig. 5. Distribution of Gauges; Beam and Column Details.

Column	Section	Flange Plate	Concrete Encasement
G 18	$12'' \times 8'' \times 65$ lb	$2/16'' \times 2\frac{3}{4}''$	$24'' \times 21''$
G 30	$12'' \times 8'' \times 65$ lb	$2/16'' \times 2\frac{3}{8}''$	$24'' \times 21''$
G 22	$12'' \times 8'' \times 65$ lb	$2/16'' \times 2\frac{1}{2}''$	$24'' \times 21''$
G 24	$12'' \times 8'' \times 65$ lb	$2/16'' \times 2\frac{1}{4}''$	$24'' \times 21''$
G 26	$12'' \times 8'' \times 65$ lb	$2/16'' \times 2\frac{3}{8}''$	$24'' \times 21''$
E 18	$10'' \times 8'' \times 55$ lb	$2/14'' \times 2''$	$20'' \times 20''$
E 20	$10'' \times 8'' \times 55$ lb	$2/14'' \times 2''$	$20'' \times 20''$
E 22	$10'' \times 8'' \times 55$ lb	$2/14'' \times 2''$	$20'' \times 20''$
E 24	$10'' \times 8'' \times 55$ lb	$2/14'' \times 2''$	$20'' \times 20''$
E 26	$10'' \times 8'' \times 55$ lb	$2/14'' \times 2\frac{1}{8}''$	$20'' \times 20''$
M22	$12'' \times \frac{5}{8}''$ web	$2/15'' \times 1\frac{1}{8}''$	$20'' \times 20''$
J 22	$12'' \times \frac{5}{8}''$ web	$2/15'' \times 1\frac{1}{8}''$	$20'' \times 20''$

Slabs and Reinforcement

Spine level 2 $8\frac{1}{2}''$ slab
$\frac{5}{8}''$ dia. bars at $6''$ c's parallel to E 221/2
$\frac{3}{4}''$ dia. bars at $6''$ c's parallel to E 220/2
Portals level 4 $9\frac{1}{2}''$ slab
$\frac{3}{4}''$ dia. bars at $7\frac{1}{2}''$ c's parallel to J 221/4
$\frac{3}{4}''$ dia. bars at $6''$ c's parallel to J 220/4
Portals level 2 $9\frac{1}{2}''$ slab
$\frac{3}{4}''$ dia. bars at $6''$ c's parallel to J 221/2
$\frac{3}{4}''$ dia. bars at $4\frac{1}{2}''$ c's parallel to J 220/2
Slab reinforced by 4 No. r.c. beams spanning parallel to J 221/2 overall size $25\frac{1}{2}'' \times 7''$ reinforced by 2 No. $\frac{1}{2}''$ bars.

by precise levelling to a datum sufficiently far from the site to be unaffected by building operations.

Temperatures at ten points in the structure were measured by resistance thermometers embedded in the concrete. As a check on the resistance thermometers, access tubes were provided to enable a tip actuated mercury thermometer to be passed through the concrete encasement to the steelwork.

Live loading consisted of 120 oil drums filled with bricks which were to be used in the building. The drums, which weighed about 0.25 ton each, were placed in position by fork lift trucks. Live loads were applied at level 2 in the spine and at levels 2 and 4 in the portal structure. Fig. 6 shows a bay of the main spine carrying a load of 30 tons.

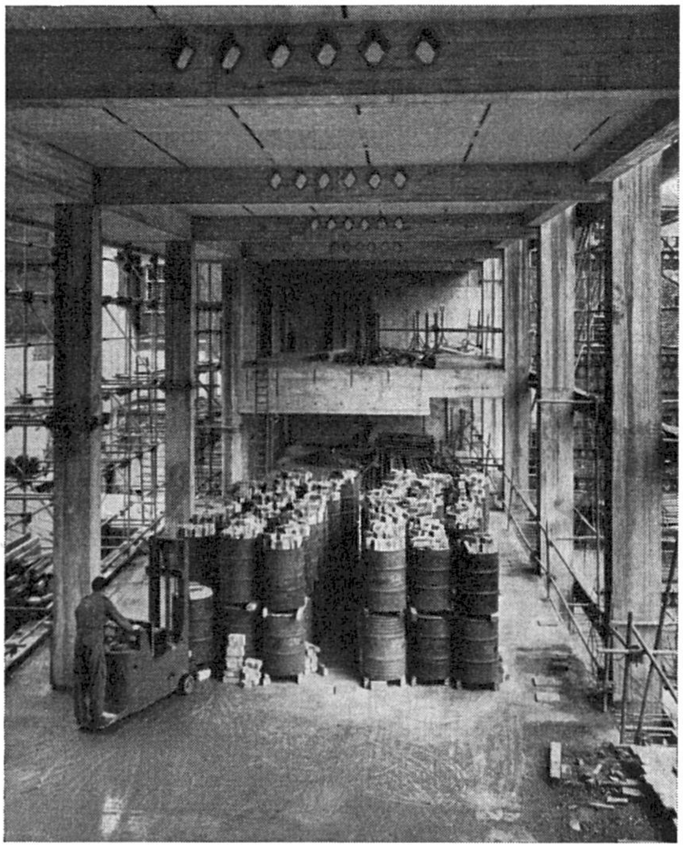


Fig. 6. Test Load 5, Main Spine.

Results

Fig. 7 shows stress histories for the spine columns, the strain being measured on the flange centre lines 8 feet 6 inches above the column bases (fig. 5). The stress plotted at zero time is the stress for the steelwork fully erected and concrete cast up to level 2, whilst the final reading plotted (twelve months later) corresponds to the structure nearing completion. An erection calendar is as follows:

Main Spine

5 December	1957	Gauges 1—20 fixed to columns.
12 December	1957	Gauges 21—62 fixed to beams.
8 January	1958	Steel erected to level 2.
13 January	1958	Steel erected to level 4.
27 January	1958	Steel erected to level 6; columns 22—26 encased to level 1; welding completed to level 4.
18 February	1958	Level 1 floor slab concreted.
11 March	1958	Level 2 floor slab concreted.
11 April	1958	Shutters removed from level 2 slab; level 4 concreted with shutters supported from level 2.
May	1958	Live load tests 1—5.
2 September	1958	Concreting completed to level 9.
25 March	1959	Most walls built; no floor finish.

Portals

10 February	1958	Gauges 63—106 fixed.
14 May	1958	Steel erected and welded.
14 August	1958	Columns encased to level 2.
2 September	1958	Level 2 concreted.
		Level 4 concreted.
November	1958	Live load tests 6—9.
24 March	1959	Most walls built; no floor finish.

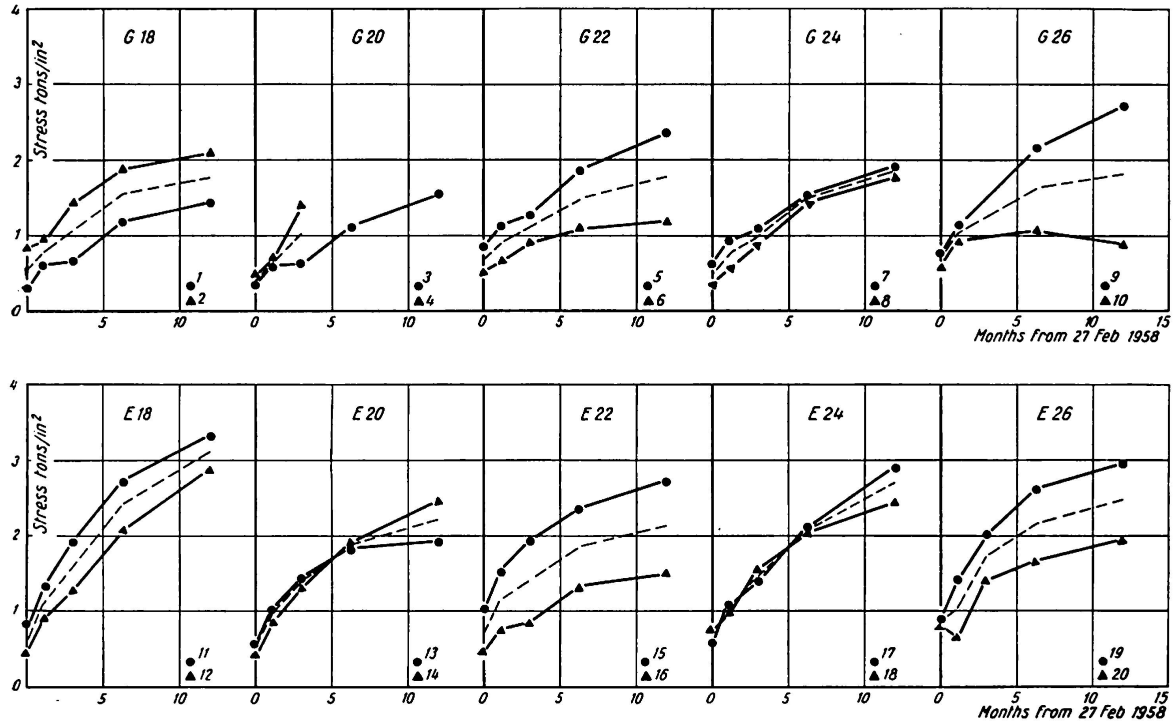


Fig. 7. Stress Histories for Spine Columns.

Note: The Location of Columns and Gauges is given in Fig. 5.

The mean stress increases as the construction proceeds, and in general the bending stress is not large. At 2 September 1958 the dead load acting on each of the columns was about 155 tons. Average steel stresses were calculated on the alternative assumptions that the steel alone carries load and that the steel and concrete act compositely. A comparison between average calculated and measured stresses can then be made:

	E. Columns tons/in²	G. Columns tons/in²
Calculated stress for steel only acting	2.15	1.49
Calculated stress for composite action, $m = 15$	1.62	1.19
Measured stress	2.07	1.52

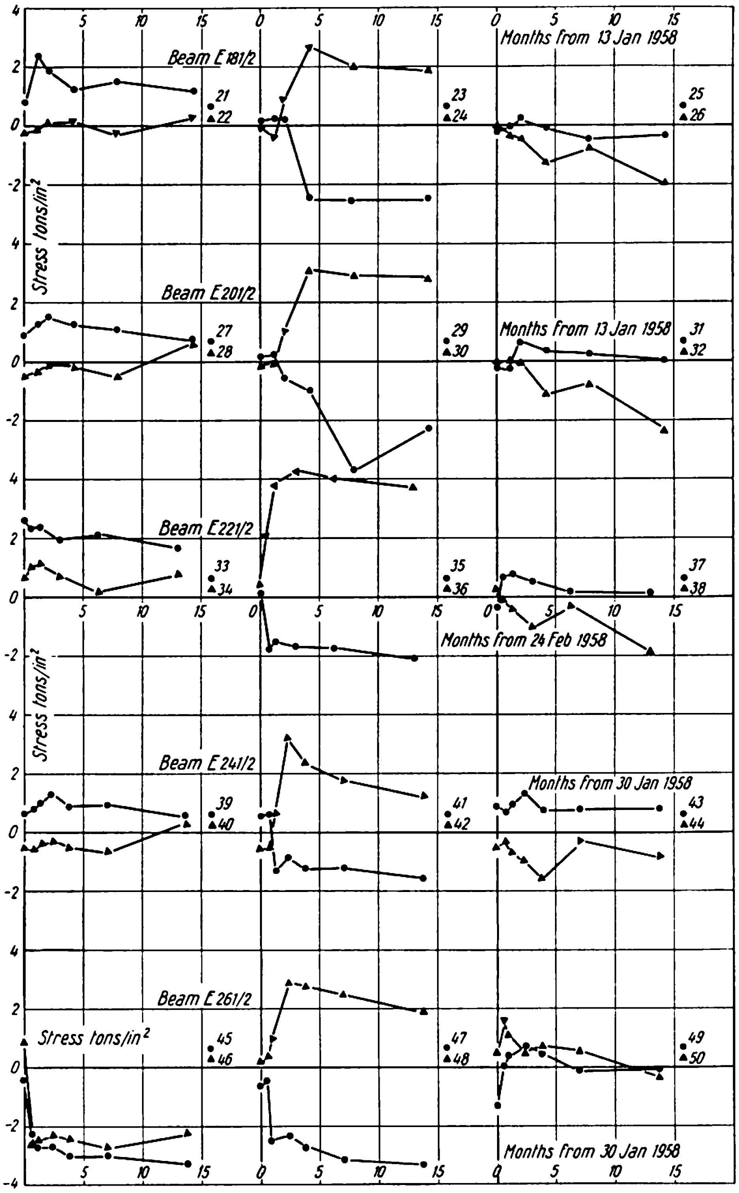


Fig. 8. Stress Histories for Spine Beams.
Note: The Location of Beams and Gauges is given in Fig. 5.

It appears from these measurements that so far the concrete has made no contribution to the direct compressive strength of the column. The gradual increase in measured stress, and the general similarity between the growth of stress in all the columns, gives confidence that the gauges are registering correctly.

Fig. 8 shows stress histories for the beams of the spine. The first readings plotted were taken when the steel frame was erected beyond level 2. Over the next four months readings were taken at different stages during erection. Approximately four months after the first readings were taken, the concrete in the test area had been placed, and the readings at this stage represent the stresses in the steel due to most of the dead load (only the floor finishes and partition walls were to be added later). Over the next ten months, when there is little change in load, there is generally little change in stress, except for a tendency for the compressive stresses in the mid beam section to increase and the tensile stresses to diminish, probably due to shrinkage. Readings near the ends of all beams except E 261/2 indicate some restraining moments, the stresses in the top flange being more tensile than the stresses in the lower flange. In most beams there is a net indicated tension or compression in the steel; gauges 45 and 46 for example indicate net compression, and this cannot be explained in terms of applied loading or shrinkage; nor indeed can it be due to the relief of locked up stresses by site welding because this had been completed before the readings were taken.

The average measured mid span stresses for the six beams due to the dead

Table I

	Compression Flange tons/in^2	Tension Flange tons/in^2
Measured stresses	1.9	3.1
Calculated stresses		
1. Steel beam only effective, simply supported ends	11.4	5.8
2. Steel beam only effective, rigidly connected to columns	4.0	2.0
3. Composite T-beam, simply supported ends, $m = 15$, concrete cracked	0.1	4.3
4. Composite T-beam, rigidly connected to columns, $m = 15$, concrete cracked	0	1.8
5. Composite T-beam, simply supported ends, $m = 15$, concrete uncracked	0.4	2.4
6. Composite T-beam, rigidly connected to columns, $m = 15$, concrete uncracked	0.2	1.1
7. Composite T-beam, simply supported ends, $m = 100$, concrete cracked	2.6	4.8
8. Composite T-beam, rigidly connected to columns, $m = 100$, concrete cracked	1.0	1.9
9. Composite T-beam, simply supported ends, $m = 100$, concrete uncracked	2.6	4.4
10. Composite T-beam, rigidly connected to columns, $m = 100$, concrete uncracked	1.0	1.8

weight of the concrete slab and encasement are compared with the stresses calculated according to various assumptions in table I.

For the composite beam calculations it has been assumed that the flange breadth is 8 feet (one third of the span). The slab weight has been taken as being distributed according to 45° lines drawn from its corners, with a triangular load distribution on the beams.

Strain measurements taken during and immediately after the pouring of the concrete have shown that the shuttering is not completely effective in carrying the weight of the concrete, and load which is transferred to the beam at this stage will be carried by the steel alone and not by the composite T-beam. On the other hand, if after removal of the shutters the whole dead load were carried by the steel alone, then the compressive stress would be nearly twice as high as the tensile stress, and this is not the case. The behaviour is further complicated by the joint rigidity (which probably increases as the concrete hardens) and by shrinkage (which induces compression in the top flange). In these particular beams the behaviour under dead load is to some extent expressed by assuming an increased value of modular ratio (such as $m = 100$) and some degree of joint rigidity; when the shutters are removed the actual value of m is of course much smaller.

Stress histories for the portal columns and beams are shown in figs. 9 and 10. The average calculated and measured axial stresses in the lower storey of columns M 22 and J 22 are:

	tons/in^2
Calculated stress for steel only acting	1.6
Calculated stress for composite action, $m = 15$	1.1
Measured stress	1.3

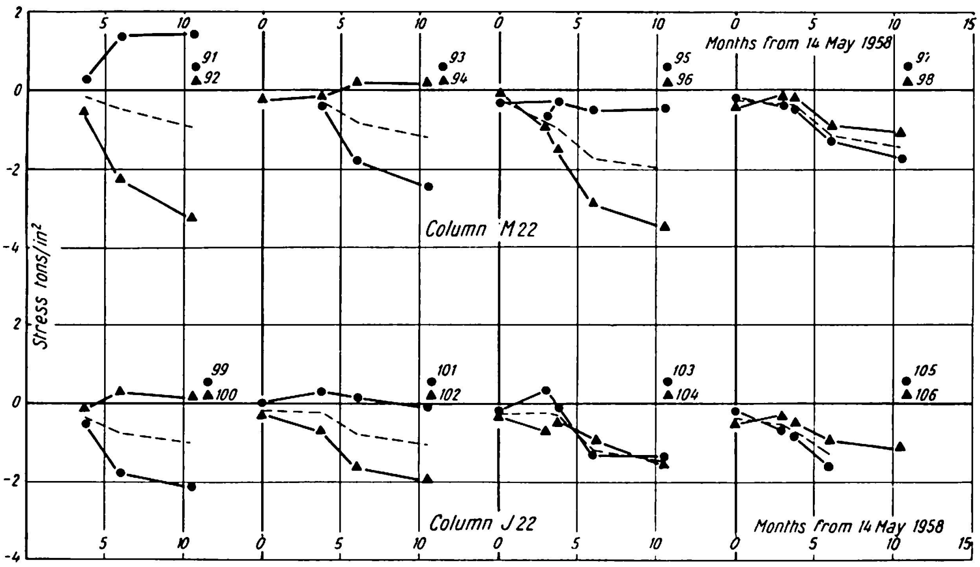


Fig. 9. Stress Histories for Portal Columns.

Note: The Location of Columns and Gauges is given in Fig. 5.

The measured mean stress at gauges 95 and 96 is higher than at 97 and 98, although the applied load is slightly smaller.

A comparison with calculated stresses of the average measured midspan beam stresses due to dead load for the three portals is:

Table II

	Compression Flange tons/in²	Tension Flange tons/in²
Level 4		
Measured stresses	0.1 (tension)	3.9
Calculated stresses		
1. Composite T-beam, $m = 15$, concrete cracked	0.1	3.9
2. Composite T-beam, $m = 15$, concrete uncracked	0.4	2.6
Level 2		
Measured stresses	0.9	3.7
Calculated stresses		
1. Composite T-beam, $m = 15$, concrete cracked	0.3	3.2
2. Composite T-beam, $m = 15$, concrete uncracked	0.3	2.2

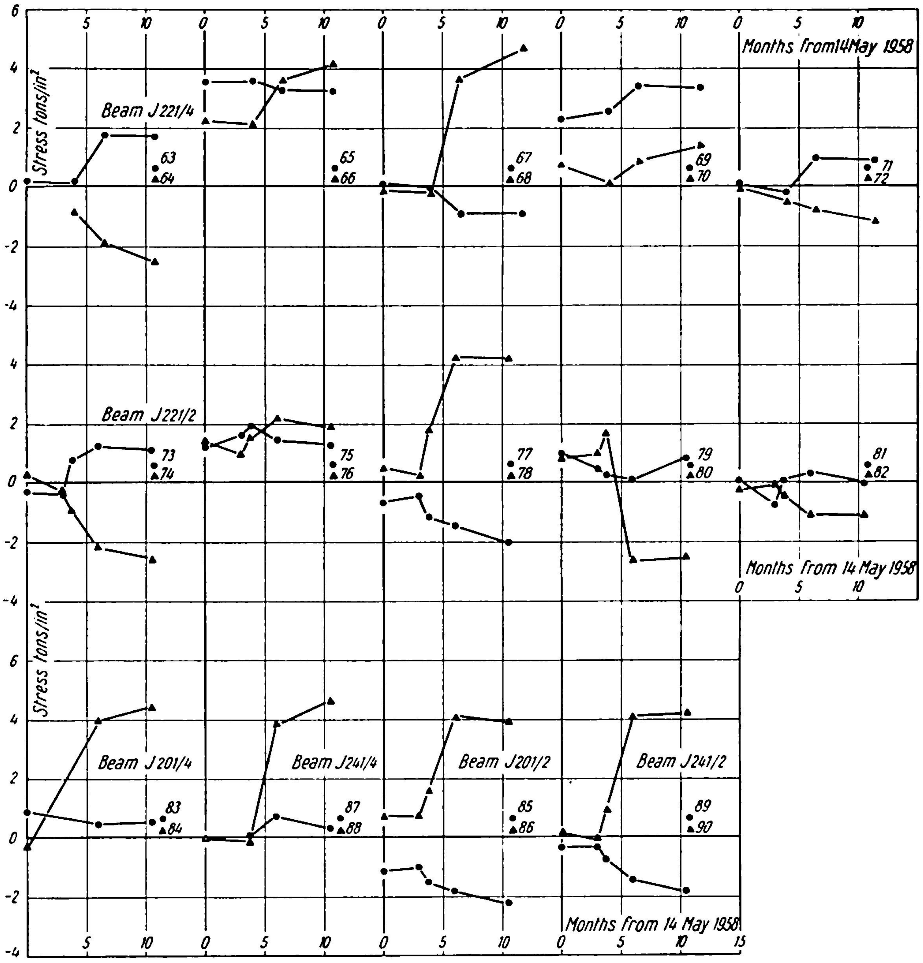


Fig. 10. Stress Histories for Beams in Portal Structure.
Note: The Location of Beams and Gauges is given in Fig. 5.

In the portal beams the compression flange stress is small, indicating that the dead load is carried compositely.

There is little evidence of bending near the feet of the portals (gauges 97, 98, 105 and 106). There is less bending at levels 4 and 2 in column J 22 than in M 22 (compare curves for gauges 99 to 104 with gauges 91 to 96) and there is less restraint at the ends of the beams connected to stanchion J 22 than to stanchion M 22 (compare curves for gauges 71, 72, 81 and 82 with 63, 64, 73 and 74). The reason for this lack of symmetry in behaviour is not apparent.

The first points plotted are for the steelwork erected and welded, but with no concrete placed. It is noticeable that considerable erection and welding stresses can be produced. The stresses at gauges 65, 66 and 69 for example, which occur near site welded connexions, were over 2 tons/in^2 before any concrete was placed.

The uniformity of the mid span tensile stresses in the six portal beams (gauges 84, 68, 88 and 86, 78 and 90) gives confidence in the measurements taken.

Table III

Main Spine, Live Load Tests

	Tests 1 and 2		Tests 3 and 4		Test 5	
	Tension Flange tons/in^2	Deflexion ins	Tension Flange tons/in^2	Deflexion ins	Tension Flange tons/in^2	Deflexion ins
Measured	0.53	0.014	0.71	0.020	0.80	0.020
Calculated						
1. $m=15$, concrete cracked	0.53	0.017	0.85	0.024	1.26	0.037
2. $m=5$, concrete cracked	0.50	0.014	0.80	0.019	1.19	0.029
3. $m=15$, concrete uncracked	0.32	0.008	0.52	0.012	0.77	0.018
4. $m=5$, concrete uncracked	0.17	0.006	0.28	0.008	0.42	0.013

Portal Frames, Live Load Tests

	Test 6		Test 7		Test 8		Test 9	
	Tension Flange tons/in^2	Deflexion ins	Tension Flange tons/in^2	Deflexion ins	Tension Flange tons/in^2	Deflexion ins	Tension Flange tons/in^2	Deflexion ins
Measured	1.29	0.058	2.42	0.084	0.84	0.037	1.32	0.060
Calculated								
1. $m=15$, concrete cracked	2.81	0.124	4.46	0.200	2.30	0.099	3.65	0.159
2. $m=5$, concrete cracked	2.46	0.091	3.92	0.148	2.06	0.075	2.99	0.115
3. $m=15$, concrete uncracked	1.88	0.097	3.00	0.157	1.60	0.080	2.53	0.129
4. $m=5$, concrete uncracked	1.00	0.049	1.60	0.080	0.88	0.038	1.40	0.068

The distribution of live loading, and the meaured deflexions and stresses, are shown in fig. 11. The loads were of the same order as those assumed in design (100 lb/ft² live load plus 50 lb/ft² floor finishes); for example, the load intensity for tests 3 and 4 was 140 lb/ft².

If it is assumed that the measured deflexions and stresses due to loads on

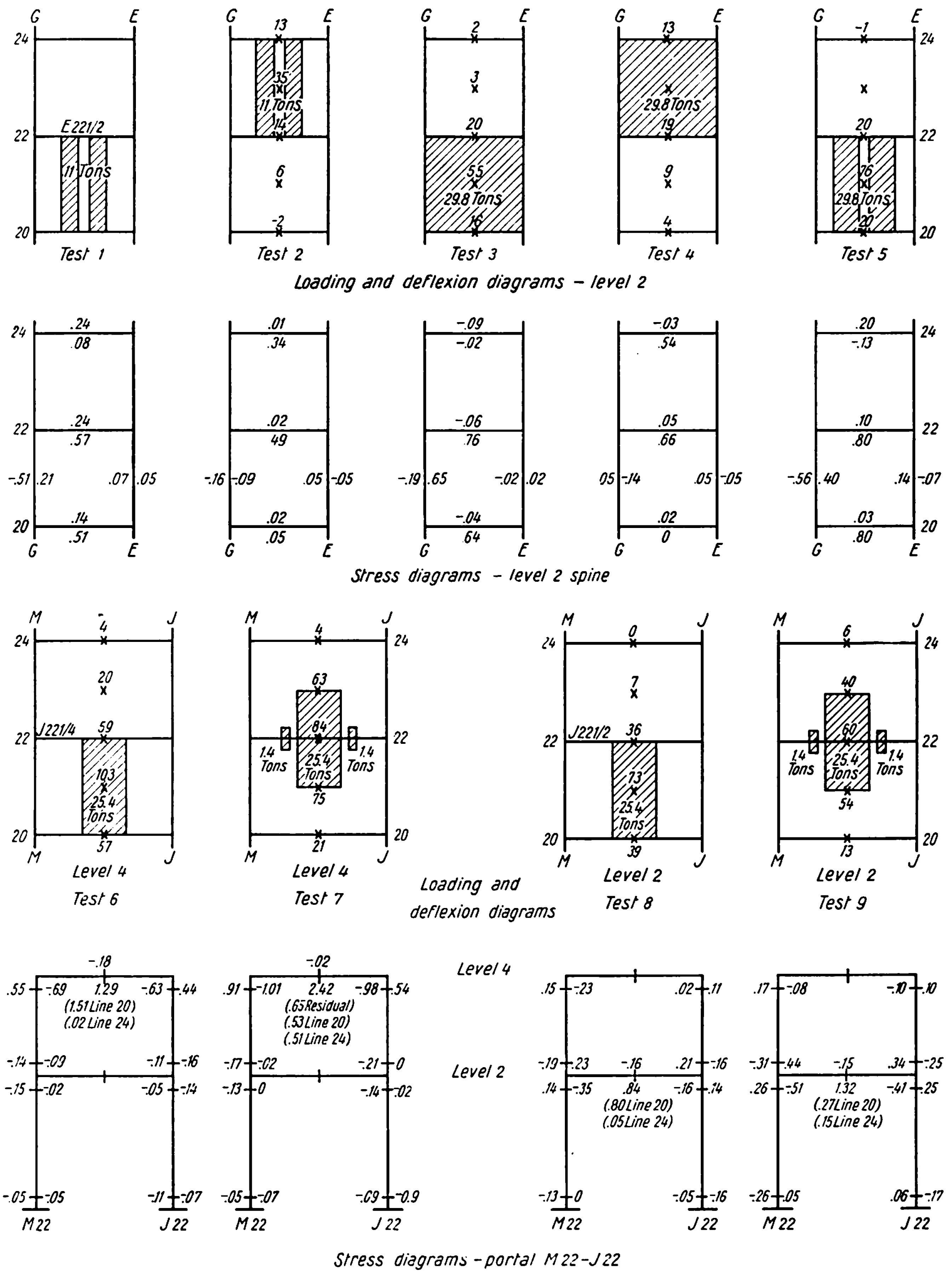


Fig. 11. Live Loading Test Results.

Notes: The Location of Beams and Columns is given in Fig. 5.

Deflexions are Inch $\times$ 10^{-3}. Stresses are Ton/Inch².

adjacent panels can be superimposed, and that loads applied on more remote panels would have no effect on the central beam (221), then the measured values can be compared with half the values calculated on the assumption that all beams are loaded (Table III). It has been assumed that composite T-beams are rigidly connected to the columns, the concrete being considered alternatively as resisting tension or as having no tensile strength. For tests 7 and 9, the calculated values have been reduced by 25% to allow for the spread of load to adjacent beams.

In the main spine the measurements are seen to be reasonably interpreted by assuming $m = 5$, the concrete having no tensile strength.

In the portals the measured values lie between those calculated assuming m to be 5 and 15, the concrete having tensile strength.

Summary of Results

1. Stresses due to erection and welding up to 3 tons/in^2 and due to dead loading up to 4 tons/in^2 were measured.
2. In the main spine columns, where heavy steel sections were used, the dead load stresses were approximately those calculated for the non composite section. In the portals, where the steel sections were lighter, there was evidence of some composite action under dead load.
3. The behaviour of beams in the main spine under dead load indicated that only part of the weight of concrete being placed was taken by the shuttering. In the portals the full dead load was taken by the composite section.
4. In the live load tests on the spine and portals the beams behaved as fully composite sections.

Acknowledgements

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Finally, they would like to express their thanks to the British Constructional Steelwork Association who greatly encouraged and helped to sponsor the work.

References

1. S. R. Sparkes and J. C. Chapman, "Some recent investigations on building structures". RILEM Symposium, October 1955.
2. J. C. Chapman, "Stud Welded Vibrating Wire Strain Gauge". The Engineer, October 24, 1958.

Summary

Experiments are described on the new Mechanical Engineering Building at Imperial College, London. The building is steel framed, with Castella beams which are designed to act compositely with the floor slabs, interaction being assured by welded stud shear connectors. Strain measurements on the steelwork were made as the construction proceeded and also during live load tests.

Dead load stresses in the columns corresponded to those calculated on the assumption that the concrete encasement was inoperative or partly operative. The action of the beams was partly composite under dead load and wholly composite under live load. Erection and welding stresses up to 3 tons/in^2 were measured.

Résumé

Les auteurs décrivent les essais qui ont été effectués sur le nouveau bâtiment de Mécanique Industrielle de l'Imperial College, à Londres. Il s'agit d'un ouvrage à charpente métallique, avec poutres à âme découpée projetées comme poutres mixtes. La coopération entre les poutres et les dalles de plancher est assurée par des chevilles soudées travaillant au cisaillement. Des mesures de contrainte ont été effectuées sur cette charpente au fur et à mesure de la construction ainsi qu'au cours des essais en charge.

Les contraintes dues au poids propre dans les poteaux correspondent aux contraintes calculées, dans l'hypothèse d'une inefficacité ou d'une efficacité partielle de l'enrobage en béton. Sous la charge du poids propre les poutres jouent partiellement en poutre mixte tandis que cette action est réalisée entièrement sous la charge de service. Les mesures ont permis de constater des contraintes de montage et de soudage atteignant jusqu'à 4,7 kg/mm^2.

Zusammenfassung

Dieser Artikel beschreibt die Versuche am neuen Gebäude für Maschineningenieurwesen des Imperial College in London. Dieses Gebäude hat ein Stahlskelett mit durchbrochenen Trägern, die mit den Decken im Verbund arbeiten. Die Verbundwirkung wird durch angeschweißte Dübel gesichert. Während der Ausführung des Baues und bei den Belastungsproben wurden Spannungsmessungen am Stahlskelett durchgeführt.

Bei Annahme, daß die Betonumhüllung nicht oder nur teilweise mitwirkte, entsprachen die Eigengewichtsspannungen in den Stützen den berechneten Werten. Die Träger arbeiteten bei Eigengewicht teilweise auch bei Nutzlast ganz im Verbund. Montage- und Schweißspannungen bis zu 3 to/in^2 wurden festgestellt.