A new method for the design of steel building frames

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A NEW METHOD FOR THE DESIGN OF STEEL BUILDING FRAMES

UNE NOUVELLE MÉTHODE DE CALCUL DES OSSATURES MÉTALLIQUES.

EINE NEUE BERECHNUNGSMETHODE FÜR STAHL-SKELETTBAUTEN.

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1. Introduction.

The methods commonly used in proportioning the members of a steel building frame bear such a slight relation to the actual behaviour of the frame that many opportunities of effecting economy of material are lost. A realisation of this impelled the British Steelworks Association, with the help of the Department of Scientific and Industrial Research, to embark, in 1929, on a complete investigation of the problem of the design of steel frames, the results of which have recently been published[1]).

Though some designers adopt other methods, regulations in most countries lay it down that design calculations may be based on the simple assumption that, when considering the effect of vertical loads, the beams are connected to the stanchions by perfect pin joints incapable of transmitting moments. This makes the choice of the necessary beam section easy. Arising from this the only load coming on to a stanchion from a beam is taken to be a vertical reaction. When the frame is called upon to withstand horizontal loads due to wind, the stresses can be calculated without difficulty if the same assumption of pin joints is made, but the resulting stresses will be found to be excessive. When studying the effect of wind loads, therefore, it is customary to consider the connections between beams and stanchions as perfectly rigid joints capable of transmitting any bending moment rather than as pin joints. The result is that the fundamental assumptions in use today are diametrically opposite and depend on the type of load considered. With these assumptions the detailed design of the beams is easy and errs on the safe side, but that of the compression members is much less satisfactory since it is usual to consider each stanchion length as a pin ended strut subjected to the eccentric reactions from the beams.

The fallacy in this method of design is obvious. Practical beam to stanchion connections, whether bolted, riveted or welded, cannot be at the same time perfect hinges and completely rigid joints; they must be in general semi-rigid, capable of providing some restraining moments at the ends of the beams and corresponding end moments on the stanchion lengths.

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[1] First, Second and Final Reports of the Steel Structures Research Committee, H. M. Stationery Office, London, 1931, 1934 and 1936.

To a limited extent cognisance is taken of this fact in recent regulations issued by some countries. Thus in Germany when certain constructional requirements are fulfilled the beam may be considered partially restrained and the maximum bending moment taken as less than that in a simply supported beam but no satisfactory definition of "partial restraint" appears to be given. According to the British Standard Specification No. 449 ("The Use of Structural Steel in Buildings") though no allowance is made for the restraint supplied to the beams, credit is allowed for the support given by the beams to a continuous stanchion, a unique clause the justification of which has been examined elsewhere [2]). In neither case is any notice taken of the appreciable end moments to which a stanchion length is subjected when the beam framing into one side only is loaded and it is usual to excuse this neglect on the grounds that the maximum bending moment will occur at the end of a stanchion length so that even if it is sufficiently large to produce yield of the material there, no danger to the safety of the member will result. The caution with which this excuse must be accepted will be shown in this paper. A consideration of the end moments applied to a stanchion length indicates another unjustifiable assumption made in the existing methods of design, namely that the most rigorous stress conditions are produced in any member when all members of the structure carry their full load.

2. Experimental Investigations.

As there appeared to be no information of any value as to the real behaviour of framed structures a considerable experimental investigation was undertaken. In addition to a comprehensive series of laboratory tests of bolted, riveted and welded connections, undertaken by Professor Batho of Birmingham University, five full sized structures were tested. Strain and deflection measurements were made, enabling the stress distribution in the members and the behaviour of the connections to be determined under loads greater, in some cases, than those which the frames were designed to carry. The first structure to be dealt with was a three storey experimental frame the members of which were all $8^{in} \times 4^{in} \times 18$ lb steel joists with storey heights 8^{ft} and beam spans 16^{ft}. The other four structures were actual buildings, the Museum of Practical Geology, South Kensington, an Hotel Building, an Office Building and a Residential Flats Building, all in London. In each case the framework was tested in its bare state and also after floors, walls and stanchion casing had been added. Space is not available here for a full description of this work but detailed accounts of the methods of test and of the results will be found in the First, Second and Final Reports of the Steel Structures Research Committee.

The more important points brought out by the experimental investigations can be summarized as follows:

a) Standard types of riveted and bolted beam to stanchion connections are capable of transmitting large bending moments, with the result that appreciable restraining moments are developed at the ends of a loaded beam in a steel framed building. The connections themselves may be treated as units and the relatively large stresses, exceeding the yield of the material, developed in them by the moments are not a source of danger.

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[2]) Baker, J. F., "A Note on the Effective Length of a Pillar Forming Part of a Continuous Member in a Building Frame", Second Report of the Steel Structures Research Committee, H. M. Stationery Office, London, 1934.

The magnitudes of the restraining moments can be gauged from the fact that the "equivalent eccentricity" of a connection (defined as the distance from the centre line of the stanchion at which the reaction from a similarly loaded simply supported beam would have to act to produce a moment equal to that observed), lay between 30.0 and 44.6 inches for the bare frame of the hotel building, reducing the maximum stresses in the beams by from 17 to 25 per cent. of those which would be found in similarly loaded simply supported beams.

b) Although there was some variation due to workmanship in the behaviour of connections made to the same design, it was possible to set out a lower limit curve for each type of connection, enabling a safe estimate of the restraining moment to be made. The degree of variation is shown in Fig. 1 where the

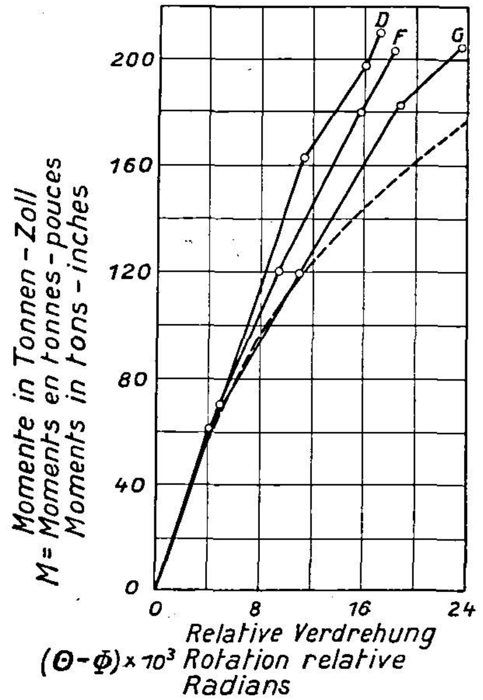

Fig. 1.

curves between M, the moment transmitted, and $(\Theta - \Phi)$, the relative rotation of the members joined by three similar connections consisting of $4^{in} \times 4^{in} \times 1/2^{in}$ flange cleats 8^{in} long and $4^{in} \times 4^{in} \times 1/2^{in}$ web cleats 10^{in} long are plotted together with the lower limit curve, shown by the broken line, deduced from the laboratory tests.

c) The moment transmitted through a connection from a loaded beam develops bending moments in the stanchion which are many times greater than those taken into account today. This will be realised from the magnitude of the actual "equivalent eccentricity", 44.6 inches, mentioned above. The value used in design would have been at most 8 inches, the stanchion being a $12^{in} \times 8^{in}$ joist.

d) The bending moments developed in a stanchion due to load applied to a beam are appreciable, not only in the stanchion lengths to which the beam is attached but also in those more remote. This is shown in Fig. 2 where the bending moment diagram is plotted for three lengths of a continuous stanchion (after the floors had been laid and the stanchion cased in brickwork) due to the application of a distributed load of 13.8 tons, or 88 lb per sq. ft. of floor area, to the floor at one side of the stanchion at level D. The form of bending in each of these stanchion lengths is designated 'single curvature' bending, the end moments having the same sense, both clockwise or both anti-clockwise. Where the end moments are of opposite sense the bending is said to be in 'single curvature'.

e) The ratio of maximum bending moments in the stanchion lengths above and below a loaded beam can be very different from that of the stiffnesses (moment of inertia/length) of the stanchion lengths.

f) The addition of floors, walls and stanchion casing to a frame does not bring about any fundamental change in the behaviour of the frame.

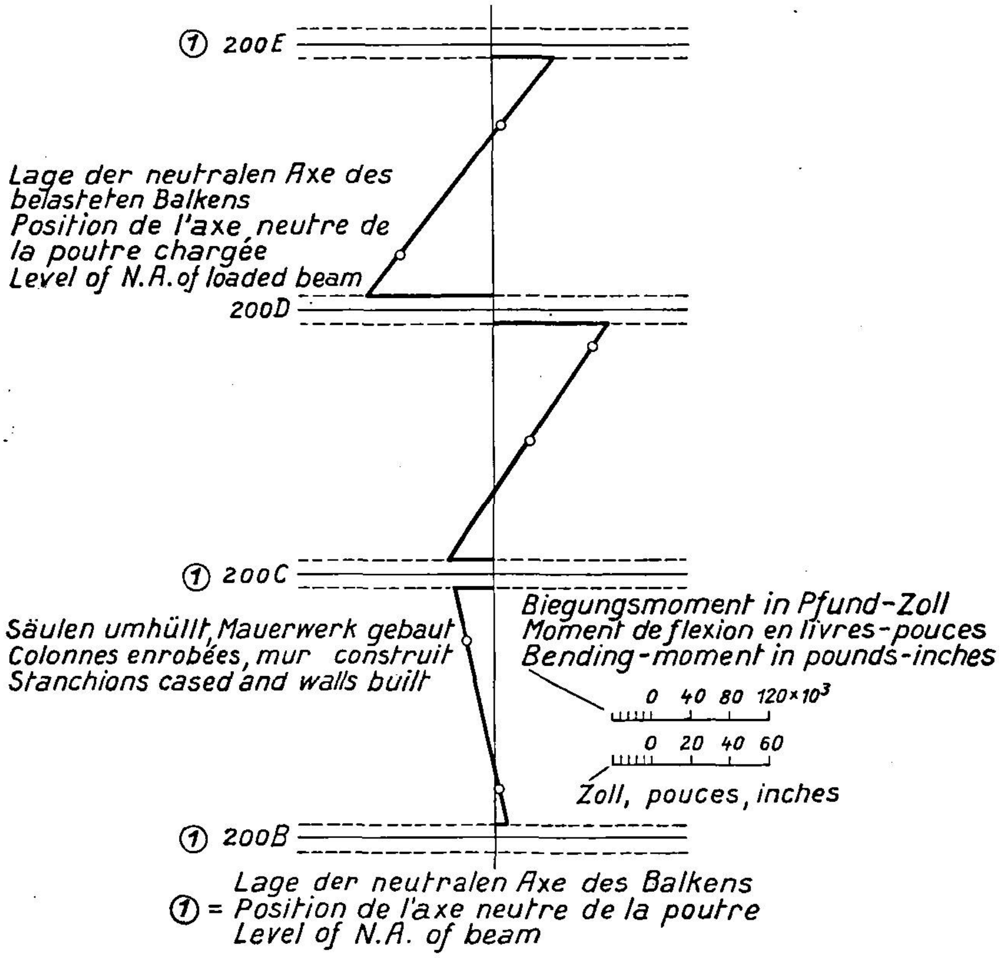

Fig. 2.

g) Concrete casing can increase the rigidity of connections appreciably in the working range.

h) The presence of clothing decreases the maximum stresses in the steel beams of a frame but it may increase the bending stresses in the steel cores of the stanchions.

i) The methods of stress analysis developed, extensions of the well-known slope deflection method taking into account the semi-rigidity of the connections, could be depended upon to give a true picture of the distribution of stress in the steelwork of a frame even when it was clothed.

The investigations thus show that the stress calculations made in design today give a very faulty representation of the distribution in the frame and, perhaps most important, disprove the assumption that the worst conditions are provided for each member when every member carries its full load.

The Steel Structures Research Committee set out to produce a method of design, simple enough for ordinary practice, based on an accurate estimate of conditions in the structure and such that an adequate load factor or ratio (approximately 2) would always be provided, that is to say the design loads could be increased in this ratio without producing a stress anywhere in the structure greater than the yield stress of the material.

3. The Design of Beams.

The tests on existing buildings show that the stresses in a member of a steel building frame are influenced not only by the size of that member and

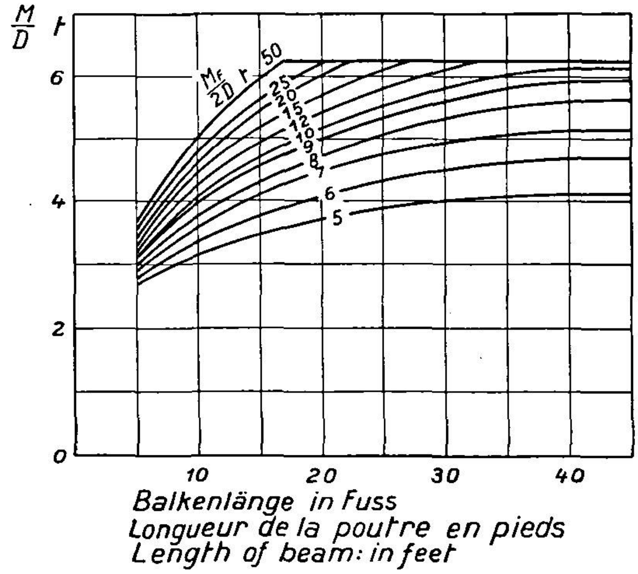

Fig. 3.
Berechnung von Balken für Verbindungen der Klasse „B".
Calcul des poutres pour des liaisons de la classe „B".
Design of beams with class „B" connections.

the load it carries, but also by the proportions and conditions of the adjacent members and of the connections between them. It is not difficult to see that the restraining moment developed at the end of a loaded beam, joined to a stanchion by a certain connection, depends not only on the characteristics of the connection but on the stiffness of the stanchion. If the stanchion is so stiff that it can be assumed to be rigid, then the restraining moment developed will have its maximum value; as the stiffness of the stanchion decreases so does the restraining moment. The maximum stress in the beam is influenced by the magnitude of the restraining moment, so that the suitability of a beam to carry a given load depends on the connections at its ends and on the stanchion to which it is attached. In the same way the bending moment applied to the stanchion by a loaded beam is influenced by the proportions of the beam.

As a designer cannot afford to make a tentative design of the whole structure and then to modify each member as the effect of the remainder of the frame is appreciated, some means had to be found which would enable one member to be designed economically without reference to the actual properties of the rest of the structure. It was found that, while the stresses in each member are affected to some extent by the conditions of the rest of the frame, the maximum stress in a loaded beam fitted with a certain semi-rigid beam to stanchion connection is not particularly sensitive to changes in the stiffnesses of the members into which it frames. A safe design without serious loss of economy would therefore result if a lower limit was assumed for the stiffnesses of these members when the restraining moment on the beam was estimated. From the evidence supplied by a large number of frames designed under existing methods it was seen that the sum of the stiffnesses of the stanchion lengths into which a beam framed would not, except in special cases, be less than two-thirds of the stiffness of the beam. Assuming the stiffness of the stanchion lengths to be two-thirds that of the beam, it was possible to find the restraining moments at the ends of any loaded beam fitted with any type of standard connection, and therefore, taking account of the relief given by these moments, to design a beam which would be safe and more economical than those used today, whatever the actual sizes of the adjacent members in the structure. In Fig. 3 are shown the curves giving the restraining moment M provided at the end of any beam of depth D when fitted with standard connections of Type B ($6^{in} \times 4^{in} \times 1/2^{in}$ flange cleats 5^{in} long) subjected to a load such that the mean of the fixed end moments (i. e. the moments present at the ends of a similarly loaded beam with completely fixed ends) is M_F. With the beams designed in this way, the design of the stanchions can be rapproached with some confidence.

4. The Design of Stanchions.

The difficulty in designing a stanchion length is to decide what conditions give rise to the greatest stress in the member. In Fig. 4 are set out diagrams showing the bending stresses in a symmetrically placed stanchion length, AB, deduced from the results of the tests on existing buildings, due to various arrangements of superimposed load on the beams. In the existing method of design it is usual to choose a stanchion section from a consideration of the conditions existing when all floors are loaded, Fig. 4 a. It is quite clear, however, that this load will not give rise to the greatest stress in the stanchion since, by removing load from the floors below on alternate sides of the stanchion, as shown in Fig. 4 b, considerable end bending moments are developed in AB. This removal does not reduce the magnitude of the end load on AB but it does appreciably increase the maximum bending stress in the stanchion length.

The conditions shown in Fig. 4 b do not necessarily develop the absolute maximum stanchion stress. If load is removed from part of the floor immediately above the stanchion length as shown in Fig. 4 c the end bending stresses are increased further and the decrease in axial end load arising from the removal of the load from the floor above may not compensate for this increase, leaving the maximum total stress greater than in Fig. 4 b. A further increase in end bending stress arises if other floors are unloaded as in Fig. 4 d, but in most structures the reduction in axial end load which follows outweighs this further increase. In these last three arrangements of load, shown in Fig. 4 b, c and d, the bending moments applied by the beams to the ends of the stanchion length bend it in "double curvature". Another condition has to be considered in which the stanchion length bends in "single curvature". An arrangement of load which brings this about is shown in Fig. 4 e together with the resulting form of the bending stress diagram. As before, the removal of further load (Fig. 4 f) increases the bending stress but decreases the axial stress.

It is impossible to say from inspection which of the arrangements of load shown in Figs. 4 b—f will give the absolute maximum stress in the stanchion length, but it is certain that one or other of them will give a greater value than the arrangement of load hitherto used (Fig. 4 a). It is clear that, for a complete treatment, the behaviour of a stanchion length under axial end load and end moments, producing both "single" and "double curvature", must be studied. The arrangement of load which must be used in design depends on the relative values of the end moments for the various cases which in turn depend on the layout and dimensions of the frame concerned.

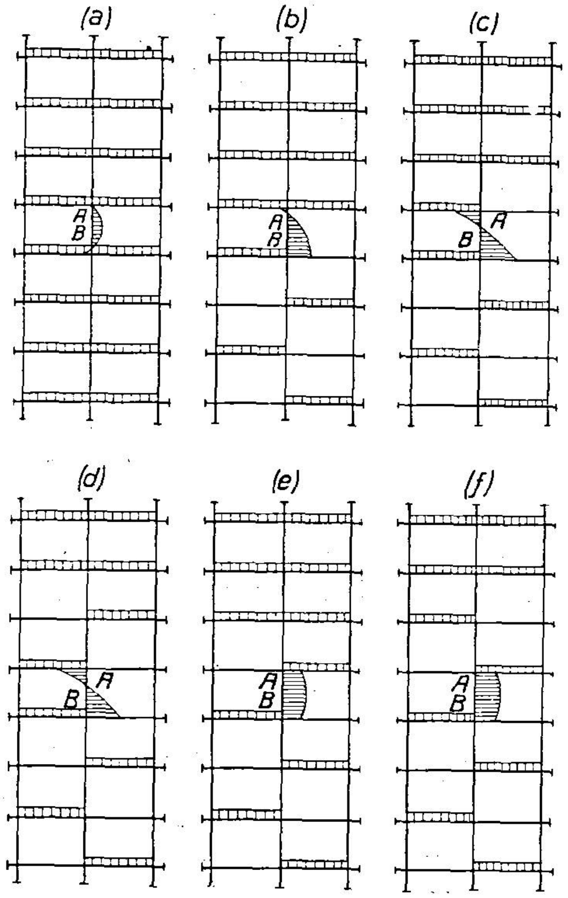

Fig. 4.

The design of a stanchion length, therefore, resolves itself into two steps, 1. the determination of the end reactions applied to the member by that arrangement of load on the structure which produces the worst conditions in the member, and 2. the estimate of the maximum stress developed in the member by those reactions, enabling the suitability of the member to be judged.

5. Critical Loading Conditions for Stanchion.

The worst possible moment that can occur in any stanchion length is made up of two parts, one due to the dead load on all the beams and the other due to the most unfavourable combination of live loads, Fig. 4. The first step in the production of a method of design is, therefore, the collection of data which will enable these moments to be estimated. The magnitudes of the moments are affected by the proportions of the members making up the frame and by the characteristics of the connections joining the members. The tests on existing buildings, while indicating that the methods of stress analysis derived earlier in the investigation were reliable, showed that the concrete casing placed around a connection could increase its rigidity very considerably. Since the end moments in a stanchion length increase as the rigidity of the connections between the members increases, it was decided that, for the normal type of clothed steel frame, it would be necessary, in estimating stanchion moments, to assume that the joints in the frame were perfectly rigid. Making this assumption, it was possible to draw up tables giving the desired information in a fairly compact form. Any exact determination of these worst moments can only be made, however, if the proportions of all the members in the frame are known. As the whole structure is not yet designed these proportions cannot be known, so that the data to be provided must be such that an upper limit value of the moment can be estimated from the meagre knowledge of the frame already possessed by the designer. All that the designer has determined at this point is the sizes of the beams. It has been shown that the moments developed in the stanchions depend on the relation between beam and stanchion stiffness, so that, if an economical upper limit value for the moment is to be found, the designer must, as he does in the existing method of design, first choose a stanchion section and then, by the methods to be described later, find whether his choice is satisfactory. The sections of the beam at one floor level and of the stanchion lengths above and below that level being known, it was found possible to compile tables giving the maximum value of the bending moment which could be applied to the stanchion at that level, no matter what the arrangement of loaded beams or the proportions and layout of more distant members in the structure might be.

The calculations which had to be made before the tables of worst moments could be compiled were arduous and somewhat complicated. It is impossible in the space available here to give any details other than specimen entries. These are shown in Tables 1 and 2.

Table 1. Total Bending Moment due to Dead Load.

$\frac{K_{BR}+K_{BL}}{K_U+K_L}$.	Bottom length; $\left(\frac{\text{total moment}}{M^D_{F_1}-M^D_{F_2}}\right)$.	Intermediate length; $\left(\frac{\text{total moment}}{M^D_{F_1}-M^D_{F_2}}\right)$.	Topmost length; $\left(\frac{\text{total moment}}{M^D_{F_1}-M^D_{F_2}}\right)$.
0.0	0.862	1.000	1.000
0.2	0.783	0.950	0.918
0.6	0.671	0.842	0 794
1.0	0.589	0.754	0.704
2.0	0.452	0.601	0.555
0 0	0.310	0.429	0 396
0.0	0.191	0.273	0.256

Each of the Tables is arranged in three main columns, referring to bottom, intermediate and topmost stanchion lengths. The moment given by Table 1 is the total moment coming on to the stanchion at B_1 (Fig. 5) when all the beams carry dead load only. $\frac{K_{BR}+K_{BL}}{K_U+K_L}$ is the ratio of the sum of the stiffnesses of the beams $A_1 B_1$ and $B_1 C_1$ to the sum of the stiffnesses of the stanchion lengths $B_2 B_1$ and $B_1 B_0$; $M^D_{F_1}$ and $M^D_{F_2}$ are the "fixed end moments" at the ends B_1 of these beams subjected to dead load. Thus when, for instance, the ratio of the sum of the stiffnesses is 2, the total moment applied to the stanchion at B_1

Table 2. Total Bending Moment due to Live Load.

$\frac{K_{BR}+K_{BL}}{K_U+K_L}$	Bottom length	Intermediate length		Topmost length	
	Double curvature	Double curvature	Single curvature	Double curvature	Single curvature
0.0	$1.149\, M^L_{F_1} + 0.287\, M^L_{F_2}$	$1.359\, M^L_{F_1} + 0.340\, M^L_{F_2}$	$1.000\, M^L_{F_1} - 0.000\, M^L_{F_2}$	$1.000\, M^L_{F_1} + 0.000\, M^L_{F_2}$	$1.000\, M^L_{F_1} - 0.000\, M^L_{F_2}$
0.2	$1.017\, M^L_{F_1} + 0.231\, M^L_{F_2}$	$1.230\, M^L_{F_1} + 0.280\, M^L_{F_2}$	$0.887\, M^L_{F_1} - 0.064\, M^L_{F_2}$	$0.925\, M^L_{F_1} + 0.007\, M^L_{F_2}$	$0.891\, M^L_{F_1} - 0.027\, M^L_{F_2}$
0.6	$08.34\, M^L_{F_1} + 0.159\, M^L_{F_2}$	$1.043\, M^L_{F_1} + 0.200\, M^L_{F_2}$	$0.731\, M^L_{F_1} - 0.110\, M^L_{F_2}$	$0.807\, M^L_{F_1} + 0.013\, M^L_{F_2}$	$0.734\, M^L_{F_1} - 0.061\, M^L_{F_2}$
1.0	$0.706\, M^L_{F_1} + 0.118\, M^L_{F_2}$	$0.904\, M^L_{F_1} + 0.151\, M^L_{F_2}$	$0.626\, M^L_{F_1} - 0.128\, M^L_{F_2}$	$0.720\, M^L_{F_1} + 0.016\, M^L_{F_2}$	$0.626\, M^L_{F_1} - 0.078\, M^L_{F_2}$
2.0	$0.516\, M^L_{F_1} + 0.065\, M^L_{F_2}$	$0.687\, M^L_{F_1} + 0.086\, M^L_{F_2}$	$0.467\, M^L_{F_1} - 0.134\, M^L_{F_2}$	$0.570\, M^L_{F_1} + 0.015\, M^L_{F_2}$	$0.462\, M^L_{F_1} - 0.092\, M^L_{F_2}$
4.0	$0.338\, M^L_{F_1} + 0.028\, M^L_{F_2}$	$0.468\, M^L_{F_1} + 0.039\, M^L_{F_2}$	$0.314\, M^L_{F_1} - 0.114\, M^L_{F_2}$	$0.407\, M^L_{F_1} + 0.011\, M^L_{F_2}$	$0.308\, M^L_{F_1} - 0.088\, M^L_{F_2}$
8.0	$0.201\, M^L_{F_1} + 0.010\, M^L_{F_2}$	$0.287\, M^L_{F_1} + 0.014\, M^L_{F_2}$	$0.192\, M^L_{F_1} - 0.081\, M^L_{F_2}$	$0.261\, M^L_{F_1} + 0.006\, M^L_{F_2}$	$0.188\, M^L_{F_1} - 0.068\, M^L_{F_2}$

due to a dead load of intensity w on the beams is equal to

$$0.601 \, x \frac{1}{12} w (l_1^2 - l_2^2)$$

where l_1 and l_2 are the lengths of the beams.

In the same way the total moment applied to the stanchion at B_1, when the live load is arranged on the beams 1. so that the stanchion length $B_0 B_1$ bends in "double curvature" [Fig. 4 d] and 2. so that it bends in "single curvature" [Fig. 4 f], can be written down from Table 2 in terms of the live loads on beams $A_1 B_1$ and $B_1 C_1$. These moments are 1, 0.687 $M^L_{F_1}$ + 0.086 $M^L_{F_2}$ and 2, 0.467 $M^L_{F_1}$ — 0.134 $M^L_{F_2}$; $M^L_{F_1}$ and $M^L_{F_2}$ being the "fixed end moments" at the ends B_1 of the beams subjected to live load. The end moments in the stanchion lengths are found by dividing these total moments between the upper and lower stanchion lengths in proportion to their stiffnesses.

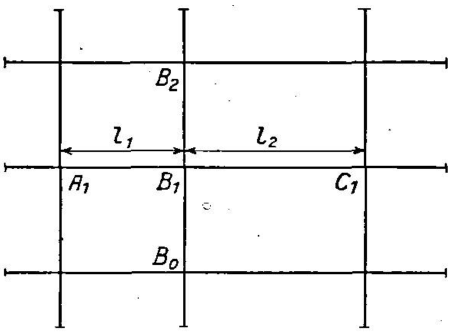

Fig. 5.

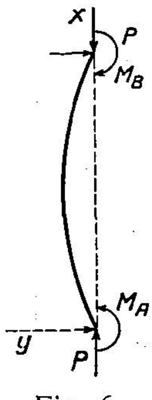

Fig. 6.

Although in compiling the Tables, such as Tables 1 and 2 to give safe values of the moments in any frome it was necessary to assume the proportions and layout of all members $\left[\text{other than those defined in the ratio} \left(\frac{K_{BR} + K_{BL}}{K_U + K_L}\right)\right]$ to be such that the greatest possible moments would be developed in the stanchion, an examination of the moments in particular frames showed that the use of the Tables does not lead to undue extravagance except in certain top stanchion lengths, where an overestimate of as much as 27 per cent. has been found.

6. Stresses in a Stanchion.

The reactions at the ends of the stanchion length are now known, and the next step is to find a convenient way of checking the suitability of the section chosen for the member. This entails finding the maximum total stress in the member or, more conveniently, demonstrating that the maximum stress does not exceed a certain permissible value. It is not difficult to derive an expression showing what must be the values of the end bending stresses (f_A and f_B) arising from end moments M_A and M_B (Fig. 6) acting in the plane in which buckling would occur, on a strut AB, subjected also to a given end load P, if the total maximum stress is not to exceed a certain permissible value p'.

This expression is

$$\frac{p' - p + f'}{f_A + f'} = \sin \alpha x \operatorname{cosec} \alpha L \left(\frac{f_B + f'}{f_A + f'} + \sin \alpha L \cot \alpha x - \cos \alpha L\right) \quad (1)$$

the value of x being given by

$$\tan \alpha x = \operatorname{cosec} \alpha L \left(\frac{f_B + f'}{f_A + f'} - \cos \alpha L\right) \qquad (2)$$

where α denotes $\sqrt{\frac{P}{EI}}$,

I „ the minimum moment of inertia of the section,
p „ P/A, A being the cross sectional area,
L „ the length of the member, and
f' „ a constant, representing the imperfections of the member.

Although these equation are too complex for use in design it is a simple matter to present the results obtained from them in the form of families of curves showing, for any ratio of f_B/f_A and therefore of M_B/M_A, the value the maximum end bending stress f_A can have without raising the total maximum stress in the

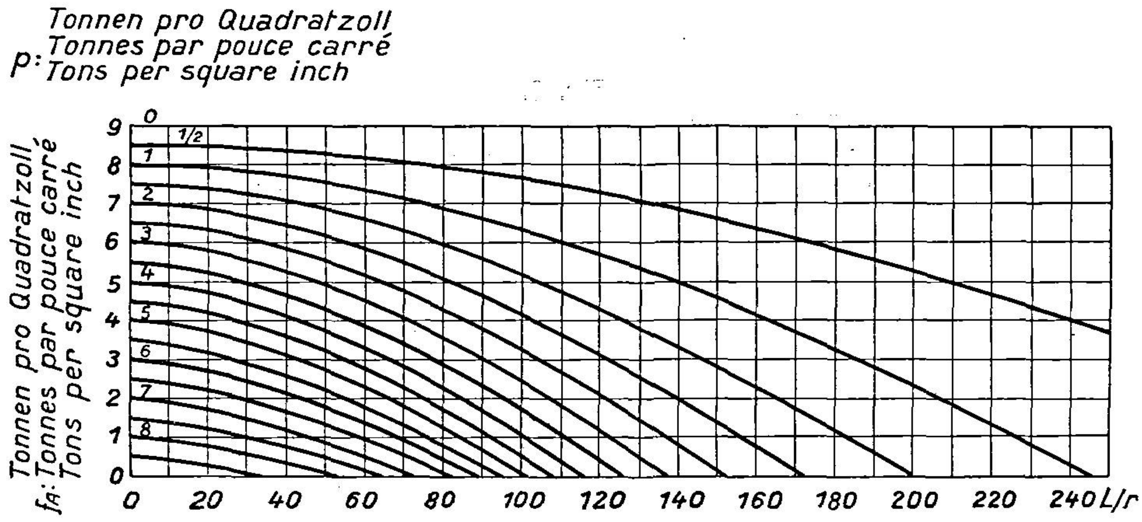

Fig. 7.
Zulässige Endspannung bei „einfacher Krümmung“.
Contrainte finale admissible pour une „courbure simple“.
Permissible end bending stress with „single curvature“.

member above the permissible value p'. When M_B and M_A are known, therefore (from Tables 1 and 2), together with P, the axial load, which is easily estimated, the suitability of the section can be tested.

This method of testing is complicated by the necessity of determining the ratio M_B/M_A, and it was thought worth while to introduce a simplification. This can be done by considering only that moment at the top end of the stanchion length which, in the type of frame under consideration, is always greater than that at the bottom end, and by assigning to the moment at the bottom end a limiting value such that safety is ensured. It is not difficult to see that in the case of "single curvature" bending the limit is given by assuming $M_B/M_A = 1$. When the bending of the member is in "double curvature" the limit is given by $M_B/M_A = -0.268$. The relevant information given by the families of curves can now be embodied in only two sets, Figs. 7 and 8, which are based on a load factor of 2 and a yield stress in the material of 18 tons per square inch. With these curves, which also make allowance for the effects of imperfections in the stanchions not included in the calculation of the end moments, and with the list of end moments given in Tables 1 and 2, a stanchion can be designed with comparative ease.

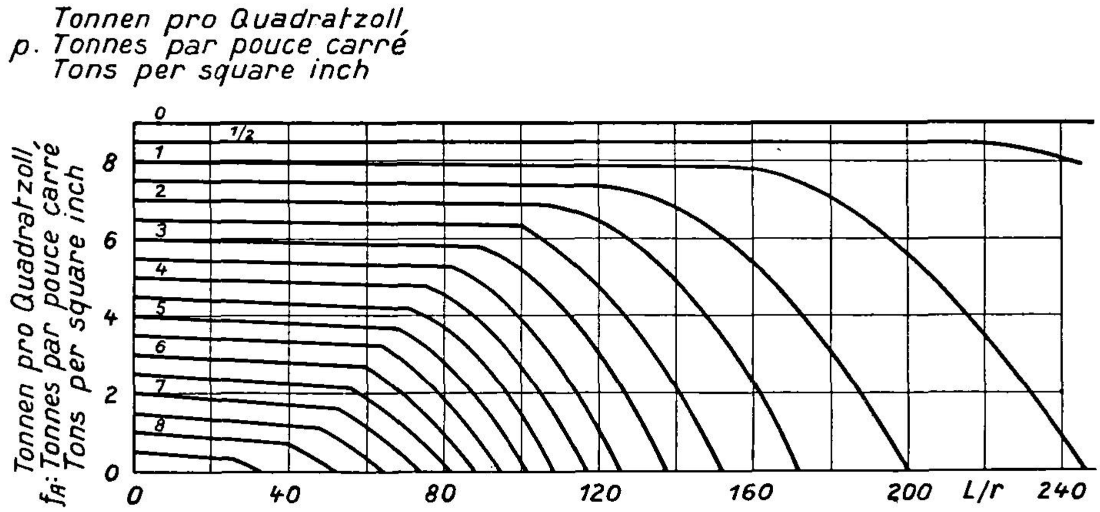

Fig. 8.
Zulässige Endspannung bei „Doppelkrümmung".
Contrainte finale admissible pour une „courbure double".
Permissible end bending stress with „double curvature".

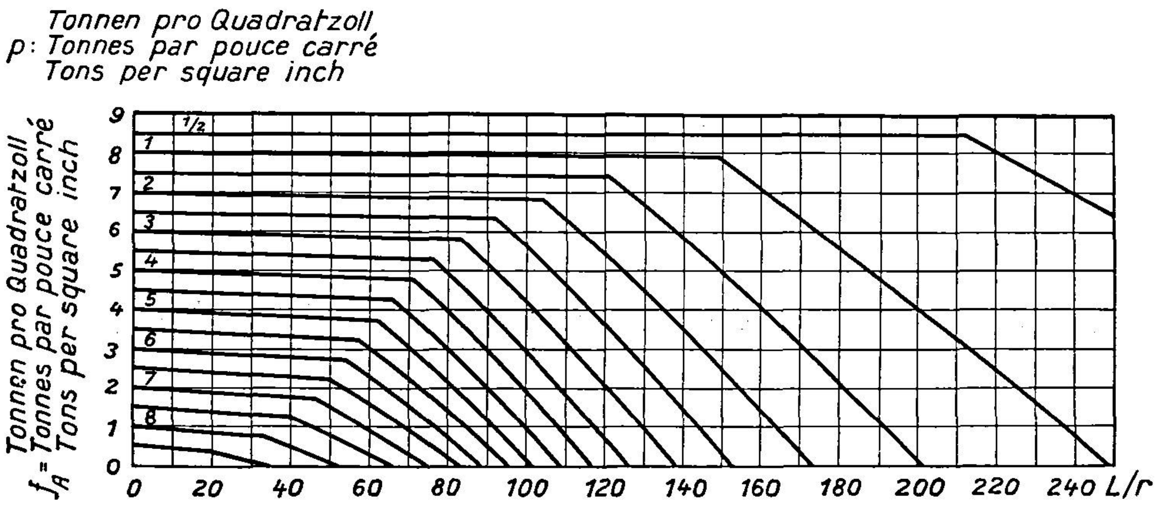

Fig. 9.
Zulässige Endspannung (zusammengesetzte Kurve).
Contrainte finale admissible (courbes composées).
Permissible end bending stress (composite curve).

7. Final Simplified Method.

It was felt, however, that in many design offices a more simplified method would be required which would not entail the consideration of two critical loading conditions.

It was necessary to arrange the data so that, while the principle of the original method would not be undermined, only one loading condition would, in effect, have to be considered.

For a symmetrical frame in which $M_{F2} = M_{F1}$, the bending moment in "single curvature" is never greater than 1/1.7 times the moment in "double curvature". If, therefore, the ordinates of the "single curvature" curve (Fig. 7) for any value of p, the axial load per unit area, are multiplied by 1.7, then the composite lower limit curve, formed from this and the corresponding "double curvature" curve (Fig. 8) by taking that portion of each which gives the lower value of the permissible end bending stress (f_A), will, if used in conjunction with the "double curvature" moments, ensure safety in all cases, whether the critical loading is such as to produce bending in "double" or "single" curvature. Such composite lower limit curves are shown in Fig. 9.

Table 3. Total Bending Moment due to Dead Load.

$\frac{K_{BR}+K_{BL}}{K_U+K_L}$.	Bottom length; $\left(\frac{\text{total moment}}{M_F^D}\right)$.	Intermediate length; $\left(\frac{\text{total moment}}{M_F^D}\right)$.	Topmost length; $\left(\frac{\text{total moment}}{M_F^D}\right)$.
0	0.862	1.000	1.000
0.1	0.820	0.975	0.957
0.2	0.783	0.950	0.918
0.3	0.752	0.920	0.882
0.4	0.725	0.893	0.852
0.5	0.697	0.865	0.822
0.6	0.671	0.842	0.794
0.7	0.648	0.820	0.769
0.8	0.627	0.798	0.745
0.9	0.606	0 776	0.722
1.0	0.589	0.754	0 704
1.25	0.545	0.709	0.659
1.5	0.511	0.669	0.619
1.75	0.482	0.632	0.583
2.0	0.452	0.601	0.555
2.5	0.405	0 546	0 503
3.0	0.367	0 500	0.461
3.5	0 336	0.462	0.426
4.0	0.310	0.429	0.396
5.0	0.268	0.375	0.349
6.0	0.236	0.333	0.310
7.0	0.211	0.300	0.279
8.0	0.191	0 273	0.256
10.0	—	—	0.227
12.0	—	—	0.197
14.0	—	—	0.174
16.0	—	—	0.156

These composite strut curves are so drawn that safety is ensured provided that the moment in "single curvature" is not greater than 1/1.7 times the moment in "double curvature". It must be remembered that the curve giving the permissible end bending stress (f_A) in "single curvature" is based on the assumption that the bending moments at the ends of the stanchion length are equal. This is important since as a result, although in an unsymmetrical frame the bending moment for "single curvature" loading may be more than 1/1.7 times that due to "double curvature", the composite strut curves are always satisfactory.

Some further consideration must now be given to the moments of Tables 1 and 2. It is not easy to write down the axial load in a stanchion length resulting from the arrangement of load on the beams which produces these moments, but it is known to be less than that arising from all beams loaded. The conditions arising when a stanchion length is subjected to this full axial load, together with the end moments deduced from Table 1 or 2 are therefore more rigorous than the real worst conditions. The combination of circumstances giving rise to the latter is likely to arise but rarely in practice, so that if the rather worse conditions, full axial load and full end moments, are assumed, a satisfactory stanchion section could be produced if the load factor chosen was less than 2 (say 1.25). The stanchion, while being safe under these impossibly rigorous conditions, would have a greater load factor than 1.25 under more normal loads. Unfortunately it was considered inadvisable to set out the me-

Table 4. Total Bending Moment due to Live Load.

$\frac{K_{BR}+K_{BL}}{K_U+K_L}$.	Bottom length; $\left(\frac{\text{total moment}}{M_F^L}\right)$.	Intermediate length; $\left(\frac{\text{total moment}}{M_F^L}\right)$.	Topmost length; (total moment M_F^L). Internal stanchion	External stanchion
0	1.149	1.359	1.360	1.000
0.1	1.080	1.290	1.261	0.961
0.2	1.017	1.230	1.175	0.925
0.3	0.961	1.175	1.098	0.892
0.4	0.912	1.125	1.031	0.861
0.5	0.871	1.082	0.968	0.834
0.6	0.834	1.043	0.915	0.807
0.7	0.796	1.004	0.869	0.781
0.8	0.763	0.968	0.823	0.759
0.9	0.732	0.934	0.782	0.738
1.0	0.706	0.904	0.746	0.720
1.25	0.648	0.837	0.669	0.673
1.5	0.596	0.780	0.602	0.635
1.75	0.554	0.729	0.548	0.601
2.0	0.516	0.687	0.503	0.570
2.5	0.456	0.615	0.431	0.517
3.0	0.408	0.556	0.390	0.474
3.5	0.370	0.508	0.358	0.436
4.0	0.338	0.468	0.334	0.407
5.0	0.289	0.404	0.293	0.357
6.0	0.252	0.356	0.261	0.318
7.0	0.224	0.318	0.235	0.287
8.0	0.210	0.287	0.214	0.261
10.0	—	—	0.181	0.222
12.0	—	—	0.157	0.193
14.0	—	—	0.139	0.171
16.0	—	—	0.124	0.153

thod of design in this way; instead, it was decided to retain the load factor of 2, used in plotting the curves in Figs. 7, 8 and 9, but to follow the lead set by existing codes, and to reduce the live axial load assumed in all storeys below the topmost. If this reduction is justifiable in the live axial load, some similar reduction is justifiable in the end moments due to the live load. This was arranged by omitting all the terms in M_{F2} from Table 2, thus giving a reduction which varies with the stiffness ratio but which never exceeds 20 per cent. It must be realised that this omission of the M_{F2} term is merely a device to secure a reduction in the total moment, while at the same time simplifying the calculation of that moment. The omission does not mean that the critical arrangements of load, which form the whole basis of this work, have in any way been altered.

The moments in topmost lengths of internal stanchions and in external stanchions needed special treatment, which will not be gone into here, but it was not difficult to produce a table of amended moments suitably reduced which, if used in conjunction with the composite strut curve, brings about a very considerable saving of labour as compared with the first method described. The amended moments are given in Tables 3 and 4.

In addition to what has been described, provision had to be made for designing frames of all types. The Tables given above could only be compiled in their simple form on the assumption that all stanchion lengths in the same storey were of the same stiffness. It was found possible to give correction factors for use when this condition did not exist, by which the tabulated moments could be multiplied to give safe values of the moments in any unsymmetrical frame. A further correction factor was needed when the intensity of load on the beam framing into the lower end of a stanchion length was greater than that on the beam framing into the upper end.

8. Conclusions.

A rational method has, therefore, been produced in Great Britain which takes into consideration the full stresses developed everywhere, yet ensures that at all points other than in the actual beam to stanchion connections the yield stress of the material is not reached. In spite of this the method produces a considerable saving of material in all beams and in the stanchions of those frames, the layout of which is rational. Design methods used in Great Britain are apt to be conservative due to the system of building regulations in force. When it is realised that there is no more objection to exceeding the yield stress at the ends of the stanchion lengths in certain circumstances, than at the ends of the beams (i. e. in the connections) and the design rules are amended accordingly sweeping economies will result since it will be equivalent to raising by an appreciable amount the permissible end bending stress f_A in Fig. 9. As a result of the Steel Structures Research Committee's investigations these economies will not be produced by blindly ignoring end moments, as appears to be done in some countries today, since it will be a simple matter, using Fig. 9, to ensure that the dangerous "single curvature" condition of bending, which may produce yield at the centre of the stanchion length, is never critical.

Summary.

In 1929, as a result of representations from the British Steelwork Association, the Steel Structures Research Committee was set up to investigate the problem of the design of steel frames. This paper gives an outline of the Committees work and the new design method recommended.

In addition to laboratory tests on connections the stress distribution in the members of five full sized structures was determined. The more important points, such as the existence of appreciable bending moments at the ends of the members, brought out by the experimental investigations are summarised.

The method of designing beams, taking account of the restraining moments, is given together with the necessary curves which define adequately, for what appears to be the first time, the degree of restraint. The corresponding end moments on the stanchion lengths are taken into account in the design method and details are given of the critical loading conditions which must be considered. Attention is drawn to the fact that the maximum bending moment in a stanction length does not always occur at the end as is sometimes assumed.

This new method, though taking into account the full stress developed everywhere, ensures that at all points, other than in the actual beam-to-stanction connections the yield stress of the material is not reached. In spite of this, when comparison is made with the existing method in use in Great Britain a considerable saving of material is found in any frame the layout of which is rational.

Zusammenfassung.

Im Jahre 1929 unternahm die Kommission für Stahlbauforschung (Steel Structures Research Committee) auf Ersuchen des Britischen Stahlbau-Verbandes (British Steelwork Association) die Aufgabe, das Problem des Stahlrahmenbaues zu untersuchen. Die vorliegende Abhandlung gibt einen Abriß der durchgeführten Arbeiten und der von der Kommission empfohlenen neuen Berechnungsmethode.

Zusätzlich der im Laboratorium erfolgten Versuche an Verbindungen wurde die Spannungsverteilung an Gliedern von fünf Stahlbauten durchgeführt.

Die wichtigeren Punkte, wie z. B. das Vorkommen von beträchtlichen Biegungsmomenten am Balkenende, ermittelt aus praktischen Untersuchungen, sind zusammengestellt.

Die Methode der Balkenberechnung unter Berücksichtigung der Einspannungsmomente wurde erklärt unter Beigabe der notwendigen Diagrammkurven, welche nach Ansicht des Verfassers zum ersten Male den Grad der Einspannung wiedergeben. Die entsprechenden Momente an den Säulenenden finden ebenfalls Berücksichtigung in der Berechnungsmethode, außerdem wurden Angaben über die zu berücksichtigenden kritischen Belastungsbedingungen gemacht.

Die maximalen Biegungsmomente in Säulen kommen nicht immer, wie oft angenommen wird, an den Säulenenden vor.

Diese neue Methode, obwohl sie sich auf die voll entwickelten Spannungen stützt, nimmt an, daß für alle Punkte, nicht nur für die Verbindungen zwischen Träger und Säule, die Fließgrenze des Materials nicht erreicht wird. Trotzdem fand man durch Vergleich mit der bis jetzt in Großbritannien gebräuchlichen Methode, daß beträchtliche Materialersparnisse sich erzielen lassen an Rahmenbauten, vorausgesetzt, es handle sich um vernünftige Dispositionen.

Résumé.

En 1929 la Commission pour l'étude de la construction métallique (Steel Structures Research Committee) a entrepris, sur la demande de la Société anglaise de la construction métallique (British Steelwork Association), d'étudier le problème de la construction des cadres métalliques. Le présent mémoire donne un aperçu des travaux exécutés et des nouvelles méthodes de calcul recommandées par la Commission.

En même temps que les essais exécutés sur des assemblages en laboratoire, on a effectué des mesures de la répartition des tensions sur les éléments de cinq ouvrages métalliques.

Les points les plus importants, comme par ex. l'apparition de moments de flexion importants aux extrémités des poutres, déterminés par des investigations pratiques, sont exposés dans ce travail.

La méthode de calcul des poutres, en tenant compte des moments d'encastrement, est exposée au moyen des diagrammes nécessaires qui d'après

l'auteur, donnent pour la première fois le degré de l'encastrement. Les moments correspondant aux extrémités des colonnes sont aussi pris en considération dans la méthode de calcul. En outre, l'auteur expose les données sur les conditions de charge critique dont il faut tenir compte.

Les moments de flexion maxima dans les colonnes ne se présentent pas, ainsi qu'on l'a souvent admis, aux extrémités des colonnes.

Cette nouvelle méthode, quoiqu'elle s'appuie sur les contraintes parfaitement connues, admet que pour tous les points, et non seulement pour les assemblages entre les poutres et les colonnes, la limite d'écoulement du matériau n'est pas atteinte. Toutefois, on a trouvé, par comparaison avec la méthode utilisée jusqu'à présent en Grande Bretagne, que l'on pouvait réaliser des économies importantes de matériau dans les constructions de cadres, en admettant qu'il s'agit de dispositions convenables.

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