Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 6 (1960)

Rubrik: IIIa. Design and execution

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IIIa1

On the Lateral Buckling of Multi-Story Building Frames with Shear Bracing

Sur le flambage latéral des portiques étagés multiples munis de contreventements (shear bracing)

Über das seitliche Ausknicken eines mehrstöckigen Gebäudes mit Windverbänden (shear bracing)

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Introduction

With the trends which seem to be developing in the architectural design of tall buildings, it is likely that questions of general instability will assume greater importance than they have had in the past. In particular, the lurching, sidesway or translational mode of buckling of tall buildings may demand greater attention than it has been given in the past.

In the past, skeleton-type tall building frames have been sheathed by rather substantial walls or wall panels of masonry construction and it is quite likely that these were sufficiently stiff to brace the frame against a lurching mode of buckling. In place of the masonry envelope which, in the past, obviated or at least minimized the necessity for considering the translational mode of buckling, the present architectural trend seems to be toward the use of light and often prefabricated panels having considerably reduced shear stiffness and hence much less effective in bracing the frame against buckling in a sidesway mode. Calculations made upon some recently designed bulding frames for the sidesway mode show that the equivalent or effective column length may be as much as three stories. This is far from the one story assumption which, in the past, has been a convenient and apparently adequate basis of design.

Unbraced symmetrical frames under symmetrical loads may buckle in either the symmetrical mode, which does not involve translation of the joints, or in the anti-symmetrical mode involving lateral displacement or lurching.

However, by considering the limiting cases of unbraced frames with infinitely stiff girders and with infinitely flexible girders, it can be shown that the lurching mode will always occur at lower loads than the symmetrical case. Unbraced unsymmetrical frames will, in general, buckle in a mode involving some lateral displacement of the joints.

When bracing is provided against lateral displacement, either in the form of shear panels or supplementary bracing members, the critical loads for the lurching mode of buckling are, naturally, increased over the corresponding loads for the unbraced frame. As the stiffness of this bracing is increased continuously, the critical loads for the lurching mode will increase until, in the symmetrical case, these loads become greater than those associated with a mode which does not involve translation of the joints.

The present paper contains some results which have been obtained during a preliminary and exploratory study of the general problem.

Limiting Cases

It is of interest to determine the stiffness required in the lateral bracing to preclude the lurching mode of buckling. For our immediate purpose it is sufficient to consider a single column and, in order to establish the requirement under the most severe condition, we shall first take the case corresponding to infinitely rigid girders.

We consider a single story and we assume that the bracing force is applied in a horizontal direction at the top of the column. This corresponds essentially to a situation in which the bracing is in the form of a broad shear-resistant panel in the plane of buckling, the panel being attached to the frame only at its upper and lower edges. The equivalent arrangement is indicated in Fig. 1. The critical load of a single column in the lurching mode may be determined

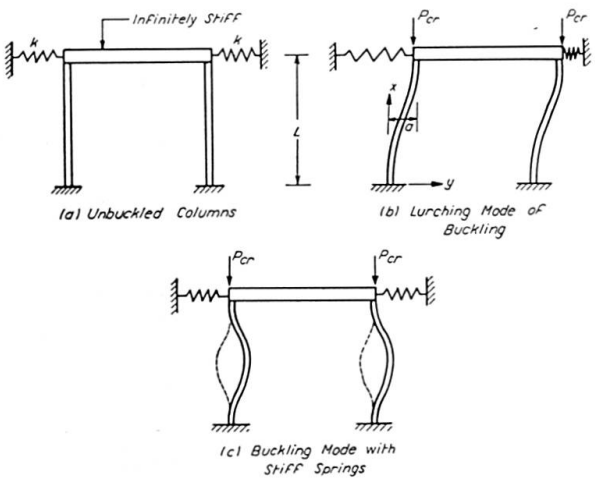

Fig. 1. Buckling Modes of Laterally Braced Columns with Infinite Rotational Restraint.

by the energy method. We take the deflections of the column as

$$y = \frac{a}{2}\left(1 - \cos\frac{\pi x}{L}\right),$$

where a is the arbitrary amplitude. Setting the bending strain energy of the column plus the extensional strain energy of the spring equal to the work done by the critical load during the buckling process leads to the stability criterion. This may be written in the form

$$\frac{P_{cr}}{P_e} = 1 + \frac{8\,k\,L}{\pi^2\,P_e}, \tag{1}$$

where

P_{cr} = critical load
P_e = Euler load $= \pi^2\,E\,I/L^2$
k = spring rate of bracing

and E is the appropriate modulus.

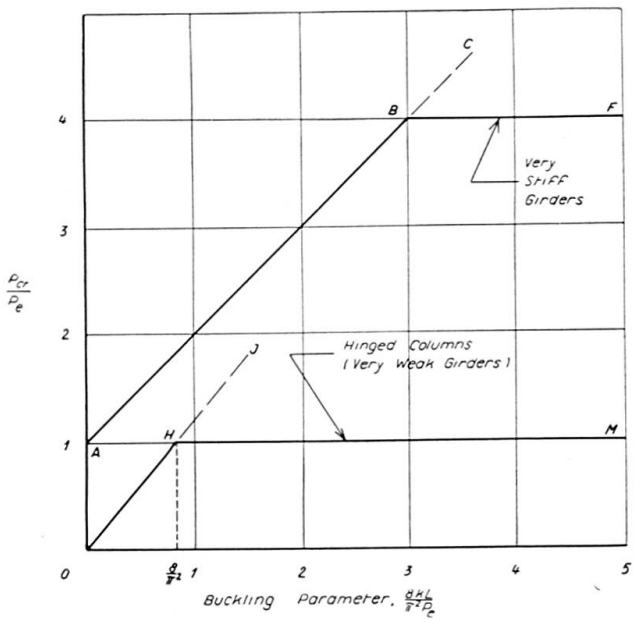

Fig. 2. Relation of Critical Load to Buckling Parameter for Limiting Cases.

Eq. (1) is plotted in Fig. 2 as the line ABC. It would appear from Eq. (1) that the critical load will increase without limit as the stiffness, k, of the lateral bracing is increased. This, however, does not follow since, under any circumstances if the column is not braced at intermediate points, the critical load cannot exceed the magnitude corresponding to the buckling mode shown in Fig. 1c. The criterion for the latter mode is shown in Fig. 2 as the line BF. It is thus seen that, in the case of very stiff girders, the column will buckle in the lurching mode when the buckling parameter has a value less than three, and will buckle without lateral displacement of its ends when the value of the buckling parameter exceeds three. That is to say, the column which is restrained by very stiff girders will not buckle in the lurching mode if

$$k > \frac{3\pi^2}{8L} P_e. \tag{2}$$

The complete criterion for the case of very stiff girders is represented in Fig. 2 as the curve ABF.

The foregoing analysis was developed for an individual column. However, certain conclusions can be drawn for the case of several columns in a given story and for complete frames. Clearly, for the collection of columns in any one story of a frame with very stiff girders laterally supported by ideal shear panels which are connected at the top and bottom of the story, the required stiffness of the lateral supports is the sum of the stiffnesses required for each of the columns, provided that each element of lateral stiffness is directly available to each column. In particular, a lurching or translational mode cannot develop in that story if

$$\sum k > \frac{3\pi^2}{8L} \sum P_e \qquad \text{(very stiff girders)} \tag{3}$$

provided that the condition on availability is satisfied. One may infer further that if the lateral support is an *ideal* (but not necessarily uniform) shear beam for the entire height of the building frame and attached only at the top and bottom of each story, the lurching mode of buckling will not develop if the total stiffness at each story satisfies Inequality (3).

In the foregoing analysis, we have considered the limiting case of a frame with very stiff girders. For comparison, we may consider the lower limiting case of a frame having girders with negligible flexural stiffness. For simplicity, we shall assume that each story segment of the multi-story column under consideration is precisely in the same state relative to the possibility of buckling; that is, the relation between axial load on each story segment and its critical load is such that no story segment either tends to support or to be supported by its neighbouring segments. This is equivalent to assuming that each story segment may be treated as hinged at each end.

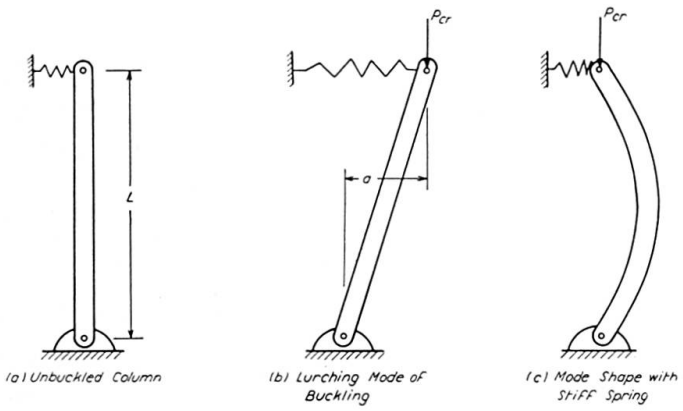

Fig. 3. Buckling Modes of a Laterally Supported Column with Negligible Rotational Restraint.

In the lurching mode of buckling shown in Fig. 3b, no bending occurs and it can be shown that the stability criterion for this mode is

$$P_{cr} = k\,L$$

or

$$\frac{P_{cr}}{P_e} = \frac{k\,L}{P_e}. \tag{4}$$

Eq. (4) is plotted in Fig. 2 as the line OHJ. As in the case of extremely stiff girders, this equation implies that the critical load increases with increasing stiffness of the lateral support. However, when the critical load reaches or exceeds the magnitude of the Euler load computed with the appropriate modulus, the column will buckle in the mode shown in Fig. 3c. Thus, the line $P_{cr}/P_e = 1$ is an upper limit to the buckling strength of the column and is shown as AHM in Fig. 2. Hence, the complete criterion for the case of very flexible girders is represented in that figure as the curve OHM.

As in the previous case, the stiffness which is required in the lateral supports for the collection of columns in any one story of a frame is the sum of the stiffnesses required for each column. In particular, a lurching mode cannot develop in that story if the spring rates of the lateral supports are such that

$$\sum k > \frac{1}{L} \sum P_e \qquad \text{(hinged columns)} \tag{5}$$

provided again that each element of lateral stiffness is directly available to each column.

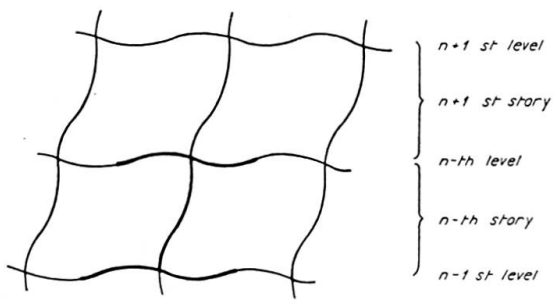

Fig. 4. Portion of Frame in Lurching Mode.

If the premises or assumptions upon which the foregoing analyses were based valid, the critical loads for each column segment in an actual frame would fall in the region MHOABF of Fig. 2. However, while the curve ABF is a reliable upper bound on the critical loads, it cannot be said that the curve OHM is an equally reliable lower bound. The assumption of very stiff girders in the first case prevented propagation of buckling deformation from story to story without violating continuity. An analogous uncoupling was assumed in the second case so that the column segments could again be treated individually. Instead of merely assuming that the girders are infinitely weak in flexure, the second case corresponds to assuming that the column segments

are hinged at the joints. In real structures, however, the columns are continuous and the segments cannot be treated on this individual basis. Curve OHM is not an entirely dependable lower bound on the critical loads.

In the case of frames with infinitely stiff girders, we were able to establish a value for the stiffness of the lateral bracing at which bifurcation of the buckling modes is possible. When, in such frames, the stiffness of the bracing is less than this value, the frame will buckle in the lurching mode; and when the stiffness of the bracing exceeds this value, the stories will tend to buckle in a "symmetrical" mode not involving translation. Although the development postulated infinitely stiff girders, this case is of more than academic interest since the results which have been obtained might serve as a basis for approximate design in cases where the girders are relatively, but somewhat less than infinitely, stiff. Furthermore, it is clear that the critical or bifurcation value of the shear stiffness, $k = 3\pi^2 P_e/8L$, for the case of infinitely stiff girders is also an upper bound to the critical value of the *stiffness* for a case in which the girders are less than infinitely stiff. Thus, when the stiffness of the shear bracing exceeds the stated value, $3\pi^2 P_e/8L$, at each story but the girders are less than infinitely rigid, the frame will tend to buckle in a "symmetrical" mode and the lurching mode generally will not have to be considered.

One additional point is in order and may be discussed at this time. The *line* OA includes all frames, broadly speaking, for which no lateral bracing is provided. To neglect any appreciable lateral bracing which actually may exist is to restrict the design to the *line* OA when, in fact, the design may lie anywhere in the area MHOABF. In such cases, if general instability is a consideration, the design may be seriously penalized as a result of neglecting the lateral bracing.

General Method of Analysis

For more accurate determination of the critical loads in cases of moderate stiffness of the girders, a more comprehensive approach must be employed. The generalized slope deflection theory may be taken as the basis for this approach. We consider a single column, continuous through the entire height of the frame and rigidly connected to the intersecting girders at each story. We may think of this column as one of the two columns of a symmetrical plane frame, and the results will be as exact as one wishes. The results will be equally exact if the column is one of a set of identical, identically loaded and identically restrained columns of a multi-bay frame; and thus, without further generalization or refinement, this approach may be used in determining approximately the critical loads of such a frame.

The displacements and loads acting upon a column segment are shown in Fig. 5. The bending moments at the top and bottom of this column segment are

$$
\begin{aligned}
M^C_{n,n-1} &= -K^C_n \left[A_n \theta_n + B_n \theta_{n-1} - (A_n + B_n) \frac{y_n}{L_n} \right], \\
M^C_{n-1,n} &= -K^C_n \left[A_n \theta_{n-1} + B_n \theta_n - (A_n + B_n) \frac{y_n}{L_n} \right],
\end{aligned}
\tag{6}
$$

where

$$
\begin{aligned}
K^C_n &= E\,I/L_n, \\
A_n &= \frac{\sin p\,L - p\,L \cos p\,L}{\frac{2}{p\,L}(1-\cos p\,L) - \sin p\,L}, \\
B_n &= \frac{p\,L - \sin p\,L}{\frac{2}{p\,L}(1-\cos p\,L) - \sin p\,L}, \\
p\,L &= \pi \sqrt{\rho}, \\
\rho &= \frac{P}{\pi^2\,E\,I/L_n^2}.
\end{aligned}
$$

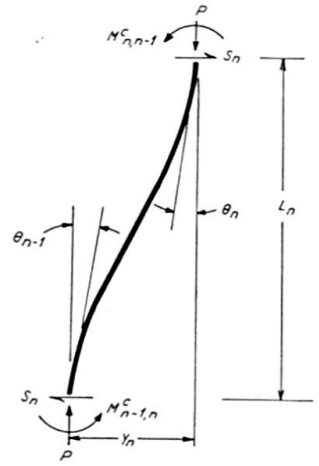

Fig. 5. Column Buckled in Lurching Mode.

Combining pertinent expressions for column moments with expressions for girder moments into a rotational equilibrium equation for a typical joint yields

$$
\begin{aligned}
K^C_n B_n \theta_{n-1} + [K^C_n A_n + K^C_{n+1} A_{n+1} + 6(1+\delta) K^G_n] \theta_n + K^C_{n+1} B_{n+1} \theta_{n+1} \\
- K^C_n (A_n + B_n) \frac{y_n}{L_n} - K^C_{n+1} (A_{n+1} + B_{n+1}) \frac{y_{n+1}}{L_{n+1}} = 0,
\end{aligned}
\tag{7}
$$

where K^G_n is the stiffness of the girder at the top of the n-th story column, the $n+1$st story is above this girder and

$$
\delta = \begin{cases} 0 \text{ when number of bays} = 1 \\ 1 \text{ when number of bays is large.} \end{cases}
$$

Taking moments about one end of the column shown in Fig. 5 leads to an expression for the transverse shear

$$
S_n = \frac{1}{L_n} K^C_n (A_n + B_n) \left(2 \frac{y_n}{L_n} - \theta_n - \theta_{n-1} \right) - P_n \frac{y_n}{L_n}. \tag{8}
$$

The shearing force in the lateral bracing system may be taken as

$$H_n = k_n y_n \tag{9}$$

where k_n is the spring rate of the shear panel at the n-th story.

Since the total shear at each story must be zero, or

$$S_n + H_n = 0,$$

the substitution of Eqs. (8) and (9) yields

$$K_n^C (A_n + B_n)(\theta_n + \theta_{n-1}) - [2 K_n^C (A_n + B_n) - P_n L_n + k_n L_n^2] \frac{y_n}{L_n} = 0. \tag{10}$$

Eliminating the y's in Eq. (7) by means of Eq. (10) yields the recursion formula

$$\begin{aligned} K_n^C (B_n - C_n F_n)\theta_{n-1} + [K_n^C A_n + K_{n+1}^C A_{n+1} - K_n^C C_n F_n \\ - K_{n+1}^C C_{n+1} F_{n+1} + 6(1+\delta) K_n^G]\theta_n + K_{n+1}^C (B_{n+1} - C_{n+1} F_{n+1})\theta_{n+1} = 0, \end{aligned} \tag{11}$$

where

$$C_n = A_n + B_n,$$

$$D_n = k_n L_n^2 / K_n^C C_n = \pi^2 k_n L_n / P_e C_n,$$

$$F_n = \frac{1}{2 - \frac{\pi^2 \rho_n}{C_n} + D_n}.$$

It may be noted that the dimensionless term D_n should be computed with the true value of $K_n^C = E I / L_n$, but all other K's may be taken as *relative* values.

Eqs. (7) and (10) or Eq. (11), written for each story, are a homogeneous set of linear algebraic equations in the lateral and angular displacements of the joints. These equations, together with the boundary conditions at the base form an eigenvalue problem in which the appropriate multiple of a prescribed pattern of column loads may be treated as the eigenvalue to be determined. There will be a number of such multiples or eigenvalues which satisfy the equilibrium equations and boundary conditions. However, only the lowest of the non-zero eigenvalues is of engineering interest.

In the case of low shear stiffness of the bracing, the frame will buckle in a lurching mode. When sufficient shear bracing is provided, the critical loads for the lurching mode may be greater than those for a "symmetrical" mode which does not involve translation of the joints. This mode involves a different bending configuration of the girders and leads to the single recursion formula

$$K_n^C B_n \theta_{n-1} + [K_n^C A_n + K_{n+1}^C A_{n+1} + 2(1+\delta) K_n^G]\theta_n + K_{n+1}^C B_{n+1} \theta_{n+1} = 0, \tag{12}$$

when the symmetrical mode is being investigated. Eq. (12) is to be written for every story. The lowest eigenvalue for this set defines the critical loads for the symmetrical mode, and comparison with the results for the lurching mode will show whether the frame will buckle with or without lurching.

Remarks on Method of Solution

Because of the highly transcendental manner in which the loads enter into the equations, ordinarily it is not feasible to extract the eigenvalues directly from the sets of equations. In rare cases, the coefficients of the displacements may have the regular character which would permit solution by difference equation methods. In other cases, a small adjustment of these coefficients may put these equations in regular form and thus permit at least an approximate solution by difference equation methods.

In the usual case, the most practical method for either desk or electronic computer will be a trial-and-error procedure in which the magnitudes of the loads are assumed. The corresponding values of A_n and B_n are then substituted into Eqs. (7) and (10) or (11) or into Eq. (12) and it is determined whether or not these equations and the base condition can be satisfied.

The solution of the equations for trial values of the loads can be obtained by any of several techniques. For example, we observe that Eq. (11) for the top level contains two unknowns, the rotation at that level and at the next lower level. We may solve this equation for the rotation at the top level in terms of the rotation at the next lower level. We use this result to eliminate the topmost rotation from the next lower equation and solve this equation for the second rotation in terms of the third from the top. We proceed in this manner, eliminating unknowns in the successive equations down to θ_2 in terms of θ_1. The rotation, θ_0, at the base is defined by a stated boundary condition and we may therefore dispose of θ_0 as an unknown in the equation for the first level above the base. With the sequential substitution and elimination of rotations, Eq. (11) for the first level above the base becomes homogeneous in θ_1. Now, the left-hand side of Eq. (11) is in fact equal to the external moment required to maintain equilibrium at the joint. Therefore, in view of the homogeneous form, if the coefficient of θ_1 vanishes, the loads form an eigenvalue set; if the coefficient is positive the frame is stable in the configuration which has been developed; if the coefficient is negative, the frame is unstable in this configuration.

Other procedures are, of course, available for effecting a solution of the set of equations. However, space does not permit a more general discussion at this time.

It may be remarked that, if an electronic computer is available, it becomes feasible to handle the exact problem of a multi-story frame having several spans. To formulate the larger problem, Eqs. (7), (10), (11) and (12) are generalized in a straightforward manner to include different rotations at each joint of each level. The resulting equations can be solved, with the aid of the computer, by relatively simple partitioning of the set and an external moment can again be computed as a criterion of stability or instability for a trial set of loads.

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Summary

The effect of shear bracing upon the critical loads of a multi-story building frame is discussed and formulas are presented for the critical loads of column segments in the two limiting cases of infinite girder bending stiffness and negligible girder stiffness. Equations for a more comprehensive theory founded upon the slope-deflection method are presented and comments are made upon methods of solution.

Résumé

L'auteur décrit tout d'abord l'influence des voiles de contreventement sur les charges critiques d'un portique étagé multiple. Pour les deux cas limites — rigidité de la traverse infinie et pratiquement négligeable —, l'auteur indique les formules permettant de déterminer les charges critiques des éléments de montants. De plus, il présente des équations découlant d'une théorie plus complète, basée sur la méthode des déformations et il commente quelques procédés de résolution de ces équations.

Zusammenfassung

Zunächst wird der Einfluß von schubfesten Tafeln auf die kritischen Lasten eines mehrstöckigen Rahmens besprochen. Für die beiden Grenzfälle des Trägers mit unendlich großer und mit vernachlässigbarer Steifigkeit werden die Formeln für die kritischen Lasten von Stützenabschnitten angegeben. Ebenso wurden Gleichungen für eine umfassendere Theorie, basierend auf der Deformationsmethode, dargestellt und dazu einige Lösungsmethoden besprochen.

IIIa2

Über den Einfluß der Normalkraftverformungen bei Stockwerkrahmen

The Effect of the Deformations Due to Axial Forces in the Design of Multi-Storey Portal Frames

L'influence des déformations dues aux efforts axiaux dans l'étude des portiques étagés multiples

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Der Einfluß der Normalkraftverformungen der Rahmenstiele auf die Beanspruchungsgrößen mehrgeschossiger Rahmen ist nicht nur um so ausgeprägter je schlanker die Rahmen sind [1], sondern er wächst auch mit zunehmendem Steifigkeitsverhältnis der Riegel zu den Stielen. Diesen Sachverhalt zeigen die nachstehend mitgeteilten Ergebnisse, die aus einer ausführlichen Untersuchung des Einflusses der Normalkraftverformungen auf die inneren Kräfte eines zweistieligen, symmetrischen Vierendeelträgers resultieren, wie er zur Aufnahme von Windkräften im Stahlbetonskelettbau häufig vorkommt [2].

Bei der Untersuchung des an seinem Fuße fest eingespannten Vierendeelträgers gemäß Fig. 1 habe ich von einem bereits früher veröffentlichten Verfahren Gebrauch gemacht, wonach die Einzelbiegesteifigkeiten der diskreten Verbindungsriegel durch die Biegesteifigkeiten kontinuierlich angeordneter Lamellen ersetzt werden [3], [4]. Die mit Hilfe eines solchen Ersatzsystems ermittelten inneren Kräfte ergeben sich in Abhängigkeit eines Systemparameters α und eines weiteren Parameters γ, der die Berücksichtigung der Normalkraftverformungen in den beiden Stielen beinhaltet.

Im Systemparameter α tritt neben den Abmessungen l_1 (Länge der Stiele), l_2 (verformbare Länge der Riegel), a_1 (Systemlinienabstand der Stiele) und a_2 (Systemlinienabstand der Riegel) auch das Verhältnis I_2/I_1 der Steifigkeiten von Riegeln und Stielen auf. Wir erkennen aus dem Aufbau von α, daß der Wert dieses Parameters sowohl mit wachsender Stiellänge l_1 als auch mit

wachsendem Steifigkeitsverhältnis I_2/I_1 zunimmt. Wachsende Stiellänge l_1 bedeutet unter Beibehaltung aller übrigen Abmessungen Schlankerwerden des Rahmens, wachsendes Steifigkeitsverhältnis I_2/I_1 bedeutet den allmählichen Übergang vom einfachen Balken zur geschlossenen Scheibe. Der Balken ist nämlich durch den unteren Grenzwert $I_2=0$ gekennzeichnet, der mechanisch das Fehlen von Verbindungsriegeln beinhaltet, d. h. die Stiele wirken als einzelne Kragträger. Die Scheibe ist durch den oberen Grenzwert $I_2=\infty$ gekennzeichnet, der mechanisch eine starre Verdübelung der beiden Stiele beinhaltet, d. h. die Verformungen des auskragenden Gesamtsystems genügen dem Bernoulli-Navierschen Geradliniengesetz.

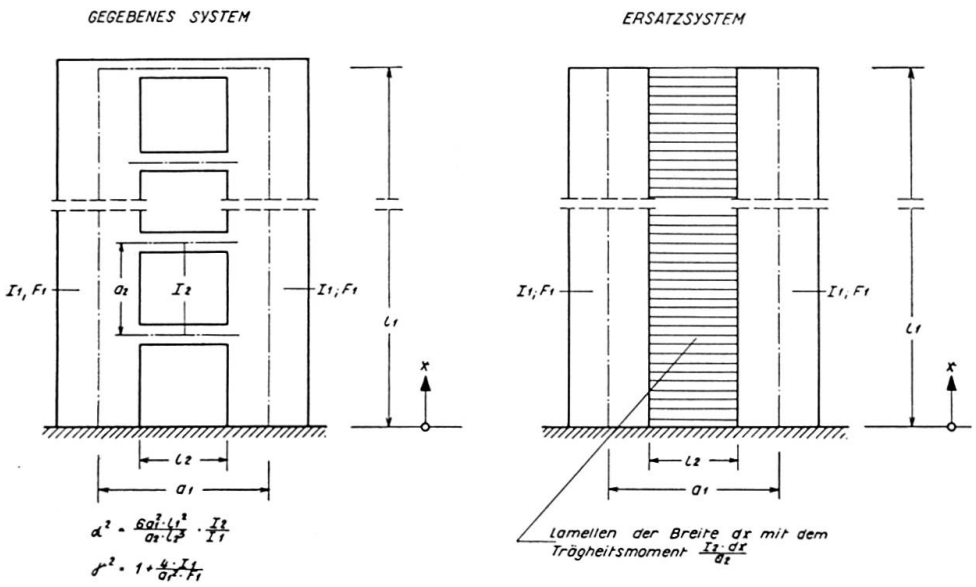

Fig. 1.

Die Berücksichtigung der Normalkraftverformungen liefert einen Wert $\gamma > 1$, ihre Vernachlässigung den Wert $\gamma = 1$, wie sich aus dem Aufbau dieses Parameters sofort ergibt, wenn die Stielquerschnittsfläche $F_1 = \infty$ gesetzt wird.

Wir wollen nun den Einfluß der Normalkraftverformungen auf die inneren Kräfte an einem symmetrischen Vierendeelträger aufzeigen, an dessen beiden Stielen eine gleich große, konstante, horizontalgerichtete Linienlast wirkt. Aus der Vielzahl der auftretenden Beanspruchungsgrößen greifen wir das Einspannmoment der beiden Stiele heraus und bezeichnen es mit

M_E bei Berücksichtigung der Normalkraftverformungen,

$M_{E,\infty}$ bei Berücksichtigung der Normalkraftverformungen und starrer Verdübelung ($\alpha = \infty$),

M_E^* ohne Berücksichtigung der Normalkraftverformungen.

Fig. 2 zeigt in Abhängigkeit von dem Systemparameter α und für verschiedene γ-Werte den Unterschied zwischen dem Einspannmoment mit und

demjenigen ohne Berücksichtigung der Normalkraftverformungen, bezogen auf das Einspannmoment mit Berücksichtigung der Normalkraftverformungen, d. h. also $(M_E - M_E^*)/M_E$. Wir erkennen, daß dieser Unterschied mit wachsendem α zunimmt. Die drei angegebenen Kurven haben die gemeinsame Asymptote 1, da $M_E^*(\alpha = \infty) = 0$ ist.

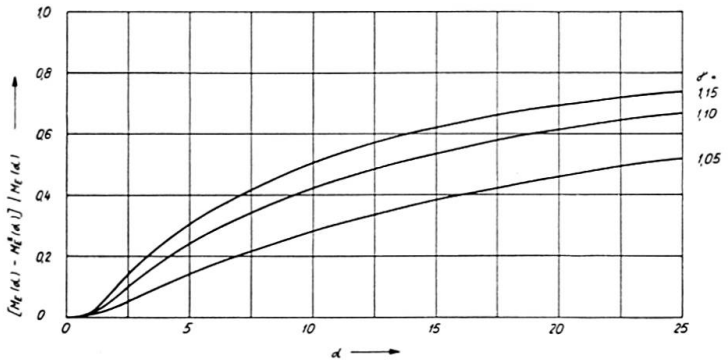

Fig. 2.

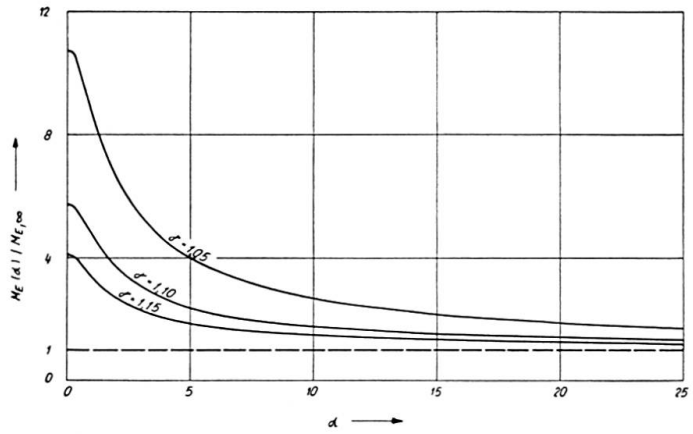

Fig. 3.

Fig. 3 zeigt in Abhängigkeit von α und für verschiedene γ-Werte das Verhältnis des Einspannmomentes bei Berücksichtigung der Normalkraftverformungen zu seinem Grenzwert für $\alpha = \infty$, d. h. also $M_E/M_{E,\infty}$. Wir erkennen aus dem Anschmiegen der drei Kurven an ihre gemeinsame Asymptote 1, wie schnell sich die Verhältnisse mit wachsendem α denjenigen des starr verdübelten Balkens, d. h. also der geschlossenen Scheibe annähern.

Fig. 4 zeigt für verschiedene Werte von α die Spannungsverteilung über die beiden Stielquerschnitte an der Einspannstelle, und zwar für den linken Querschnitt mit Berücksichtigung der Normalkraftverformungen (am Beispiel $\gamma = 1{,}05$) und für den rechten Querschnitt ohne Berücksichtigung derselben ($\gamma = 1$). Für $\alpha = 0$ ist die Spannungsverteilung in beiden Fällen gleich. Für $\alpha \neq 0$ ist ein Unterschied vorhanden, der sich mit wachsendem α vergrößert. Für $\alpha = \infty$ schließlich entspricht das linke Spannungsbild jenem der Scheibe,

während das rechte den mechanisch gegenstandslosen Verlauf zeigt, wonach das gesamte äußere Moment durch ein Kräftepaar aufgenommen wird.

Mit wachsendem α wird also der Einfluß der Normalkraftverformungen größer. Auf Grund eingehender numerischer Untersuchungen kann bei dem hier als Beispiel gezeigten Rahmensystem für $\alpha \lessapprox 5$ der Einfluß der Normalkraftverformungen vernachlässigt werden, während er für $\alpha \gtrapprox 5$ zu berücksichtigen ist. Wenn $\alpha \gtrapprox 20$ ist, so kann die Spannungsverteilung der geschlossenen Scheibe als gute Näherung benutzt werden.

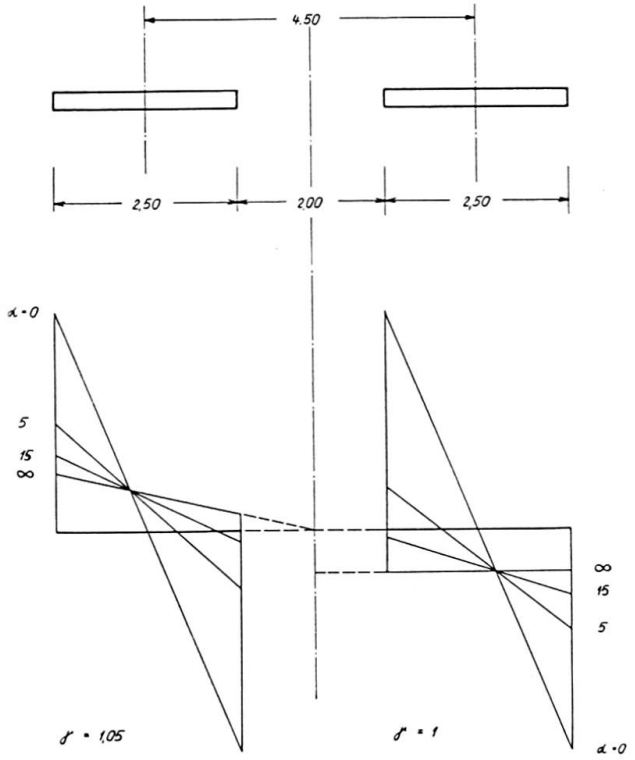

Fig. 4.

Es sei noch darauf hingewiesen, daß für mehrstielige Stockwerkrahmen analoge Verhältnisse vorliegen; dies ist ohne weiteres schon daraus zu ersehen, daß die mechanischen Grenzfälle des einfachen Balkens ($I_2 = 0$) und der geschlossenen Scheibe ($I_2 = \infty$) die gleichen sind.

Literatur

[1] P. Dubas: «Zwei theoretische Untersuchungen an mehrstieligen Stockwerkrahmen». Sechster Kongreß IVBH, Stockholm 1960, Vorbericht IIIa 2.

[2] H. Beck: «Ein Beitrag zur Berücksichtigung der Dehnungsverformungen bei Rahmen mit schlanken und gedrungenen Konstruktionsgliedern». Die Bautechnik 36 (1959), Seite 178—184.

[3] H. Beck: «Ein Beitrag zur Berechnung regelmäßig gegliederter Scheiben». Ingenieur-Archiv 26 (1958), Seite 343—357.

[4] H. Beck: «Ein neues Berechnungsverfahren für gegliederte Scheiben, dargestellt am Beispiel des Vierendeelträgers». Der Bauingenieur 31 (1956), Seite 436—443.

Zusammenfassung

Der Einfluß der Normalkraftverformung auf die inneren Kräfte eines Stockwerkrahmens wächst sowohl mit dessen Schlankheit als auch mit dem Steifigkeitsverhältnis der Riegel zu den Stielen. Dies wird für den zweistieligen, symmetrischen und am Fuße fest eingespannten Stockwerkrahmen aufgezeigt. Hierbei ist von einer vom Verfasser bereits früher veröffentlichten Berechnungsmethode Gebrauch gemacht.

Summary

The effect of the deformations due to axial forces on the stresses in a multi-storey portal frame grows with increasing slenderness and increasing ratio between the moments of inertia of the cross-members and the uprights. This is demonstrated by the calculations for a symmetrical portal frame with two uprights completely fixed at the base. These calculations were made by means of a method which the author has already described in a previous paper.

Résumé

L'influence des déformations dues aux efforts axiaux sur les sollicitations d'un portique étagé multiple est d'autant plus grande que le portique est plus élancé et que le rapport entre moments d'inertie des traverses et des montants est plus grand. C'est ce que démontrent les calculs d'un portique symétrique, à deux montants totalement encastrés à leur base. Les calculs ont été effectués à l'aide d'une méthode que l'auteur avait déjà exposée lors d'une publication précédente.

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III a 3

Comments on Semi-rigid Connections in Steel Frames

Commentaires sur l'utilisation d'attaches semi-rigides dans les ossatures métalliques

Einige Bemerkungen zu halbsteifen Verbindungen bei Stahlrahmen

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Professor of Civil Engineering, University of Michigan

Notation

P	Total load on the specimen.
E	Modulus of elasticity.
I	Moment of inertia of beam.
a	Assumed length of elastic portion.
b	Assumed length of inelastic portion.
L	$^1/_2$ span of specimen.
d	$^1/_2$ column width.
Δ	Measured vertical displacement at center.
Δ_e	Theoretical displacement at center due to deformation over elastic range.
ϕ	Angle change over one-half of inelastic range (assumed at face of column).
M	Bending moment at face of column.
Ψ	Slope of $M-\phi$ curve (assumed constant over working range).
M_{ab}, M_{ba}	End moments.
M_{Fab}, M_{Fba}	Fixed-end moments for $\Psi=\infty$.
M'_{Fab}, M'_{Fba}	Fixed-end moments for $\Psi \neq \infty$.
l	Span length of any beam.
K	$\frac{EI}{l}$.
A	$1+\frac{3K}{\Psi}$.
C_1, C_2	Coefficients of slope deflection equations.
θ_a, θ_b	End rotations.

Test Procedure for Measuring Beam Connection Properties

The following laboratory procedures and interpretation of the data has been found to be convenient and sufficiently accurate for measuring the rotational restraint of beam connections.

1. The elastic portion a of a typical test specimen as shown in Fig. 1 is assumed to extend within three inches of the edge of the connection. Strain

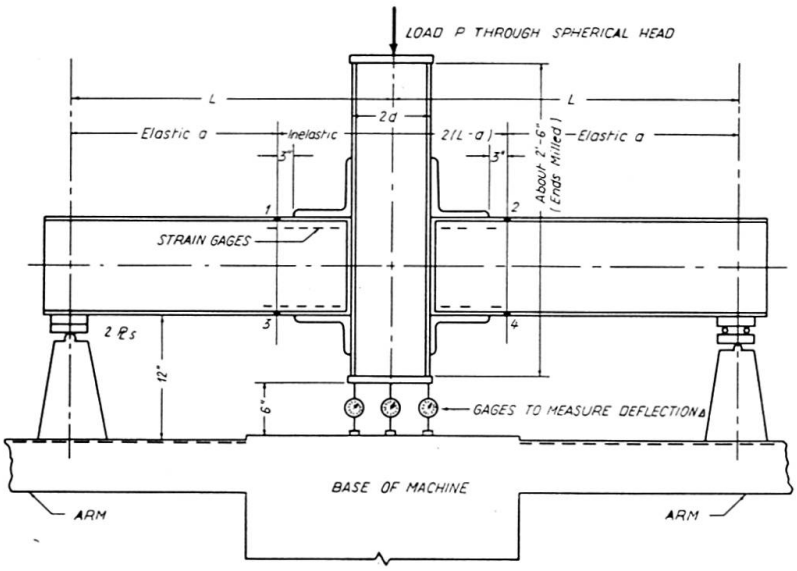

Fig. 1. Arrangement of Specimens in Testing Machine.

gage measurements have shown that between this section and the face of the column, the strain distribution across any transverse section is non-linear. This region is called inelastic although such a description is open to question.

2. The resultant angle changes ϕ, Fig. 2, for the inelastic range is assumed to occur at each face of the column for purposes of reference.

3. A laboratory test specimen as shown in Fig. 1 is therefore divided into two elastic zones a and inelastic zones $L-a$ which contain the connecting elements and column section.

4. The specimen is loaded as shown in Fig. 1 and the only measurement needed besides the central load P is the displacement Δ at the center of the span. This displacement can be measured with ordinary dial gages.

5. The numerical value of the rotation ϕ for the inelastic zone can now be determined by substracting the calculated displacement Δ_e at the center due to the strain in the elastic portions a from the total measured deflection Δ. Thus, from Fig. 2, the following relations can be established.

$$\phi\,(L-d) = \Delta - \Delta_e = \Delta - \frac{P\,a^3}{6\,E\,I}$$

or

$$\phi = \frac{\Delta - \dfrac{P\,a^3}{6\,E\,I}}{L-d}, \tag{1}$$

where

P = total load on the specimen,
E = modulus of elasticity,
I = moment of inertia,
a = length of elastic portion,
L = $^1/_2$ span of specimen,
d = $^1/_2$ width of column.

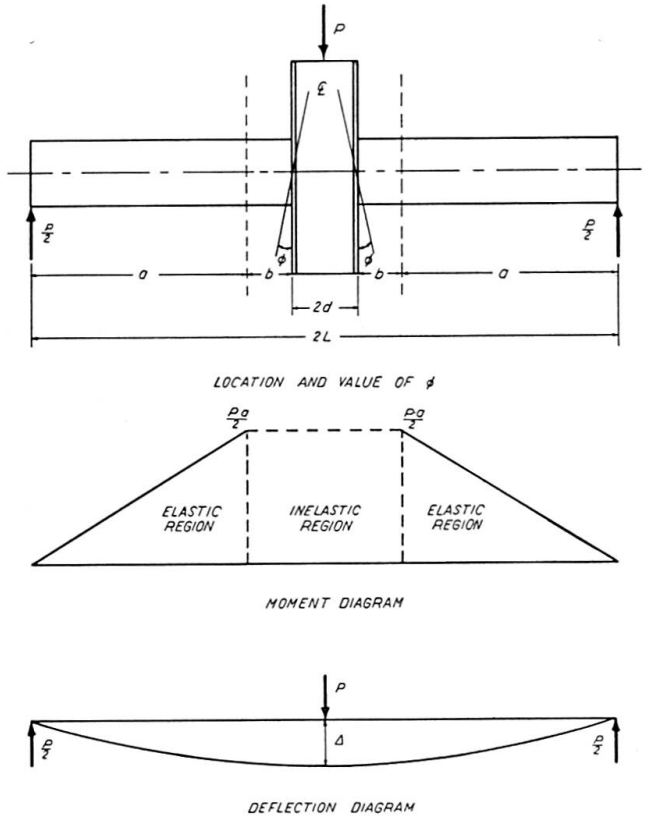

Fig. 2. Location of Angle ϕ and Displacement Δ.

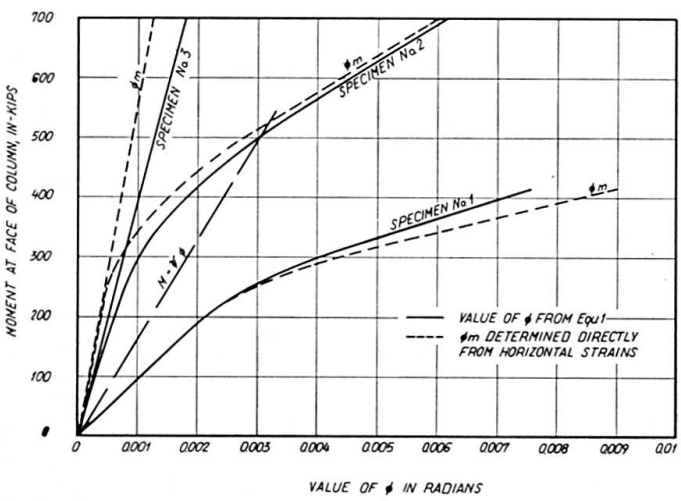

Fig. 3. $M-\phi$ Diagram Determined from Displacement Δ.

6. The values of ϕ that are determined from the measured value of Δ by means of Eq. (1), when plotted as abscissae against the moment at the face of the column as ordinate, provide typical moment-rotation (M, ϕ) curves as shown in Fig. 3. In the diagrams are also shown corresponding rotations ϕ (see broken lines) which are obtained from the horizontal movement between two reference points that were established in each flange at the edges of the inelastic zone. These gage distances are shown by points 1, 2, 3, and 4 in Fig. 1. The sum of the horizontal displacements between points 1 and 2 and between 3 and 4 divided by the vertical distance between the points was used to check the value of ϕ.

Results of Typical Tests

The test procedure as described above was used on three different specimens but, due to lack of space, only one (Fig. 4) is shown. An initial load was applied through a movable head with a spherical bearing and removed several times before the final measurements were made.

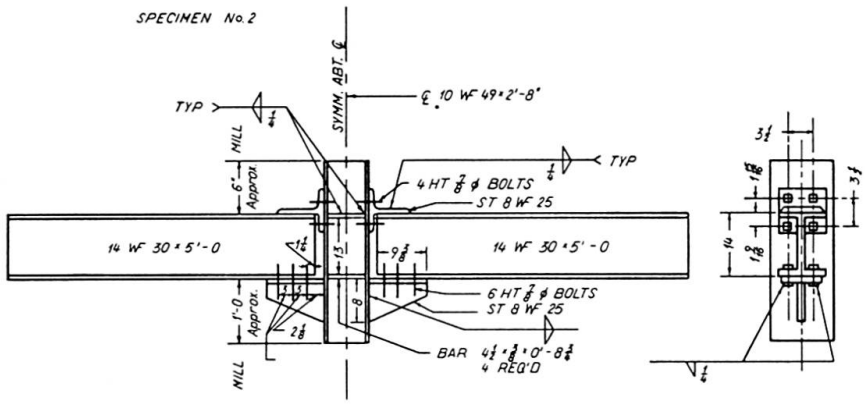

Fig. 4. Connection Details of Specimen No. 2.

The load-deflection curves for each specimen were drawn from the measured values of Δ. From such diagrams the angle change ϕ in the inelastic zone was calculated for each specimen by means of Eq. (1). The values of ϕ are shown in Fig. 3. The magnitude of the inelastic portions a is 46, 43, and 46 inches for specimens 1, 2, and 3, respectively. L is equal to 60 inches and d is 5 inches for all specimens.

Use of M, ϕ Curves in Design

If the actual M, ϕ curves in Fig. 3 are approximated by a straight line the moment M can be expressed in terms of the rotation ϕ by the relation

$$M = \Psi\phi,$$

where Ψ is the slope of the M, ϕ diagram. The quantity M/Ψ is therefore equivalent to $\frac{M\,dx}{EI}$ in a beam and can be treated as such in the calculations. In any frame where the steel girders support a reinforced concrete floor, the actual flexural rigidity EI of the beam is uncertain. However, when only steel members of constant cross-section are considered in determining the beam coefficients, the following assumptions are recommended for design calculations:

a) Consider the beam as a member with constant EI except at the ends where a concentrated angle change of M/Ψ occurs.
b) When the connection stiffness Ψ is the same for both ends of the beam, the coefficients 4 and 2, and the fixed-end moments M_{Fab} and M_{Fba} in the slope-deflection equations

$$M_{ab} = \frac{EI}{l}(4\,\theta_a + 2\,\theta_b) + M_{Fab}, \tag{3a}$$

$$M_{ba} = \frac{EI}{l}(2\,\theta_a + 4\,\theta_b) + M_{Fba} \tag{3b}$$

can be replaced by

$$M_{ab} = \frac{EI}{l}(C_1\,\theta_a + C_2\,\theta_b) + M'_{Fab}, \tag{4a}$$

$$M_{ba} = \frac{EI}{l}(C_2\,\theta_a + C_1\,\theta_b) + M'_{Fba}, \tag{4b}$$

in which, assuming that $\Psi_a = \Psi_b = \Psi$

$$C_1 = \frac{12\,A}{4\,A^2 - 1}, \tag{5a}$$

$$C_2 = \frac{6}{4\,A^2 - 1}, \tag{5b}$$

where

$$A = 1 + \frac{3\,EI}{l\,\Psi} = 1 + \frac{3\,K}{\Psi}, \tag{5c}$$

$$K = \frac{EI}{l},$$

in which l = distance center to center of columns.

Also,

$$M'_{Fab} = \tfrac{1}{6}[M_{Fab}(2\,C_1 - C_2) + M_{Fba}(2\,C_2 - C_1)], \tag{6a}$$

$$M'_{Fba} = \tfrac{1}{6}[M_{Fab}(2\,C_2 - C_1) + M_{Fba}(2\,C_1 - C_2)], \tag{6b}$$

where M_{Fab} and M_{Fba} are the usual fixed-end moments in Eqs. (3a) and (3b) that is for Ψ equals infinity and A equal to one. For a symmetrical loading such that

$$M_{Fab} = -M_{Fba},$$

then

$$M'_{Fab} = \tfrac{1}{6}[M_{Fab}(2\,C_1 - C_2 - 2\,C_2 + C_1)]$$

or

$$M'_{Fab} = \tfrac{1}{2}(C_1 - C_2)\,M_{Fab}.$$

Important Features of Semi-Rigid Connections

As variations in the coefficients C_1, C_2, M'_{Fab}, and M'_{Fba} are important factors in a structural design, it is interesting to note that a particular end connection may provide considerable restraint for a beam with a small K/Ψ value, but relatively little if the beam has a large K/Ψ value. The variation of the coefficients C_1 and C_2 with respect to K/Ψ are shown in Fig. 5. A disturbing feature

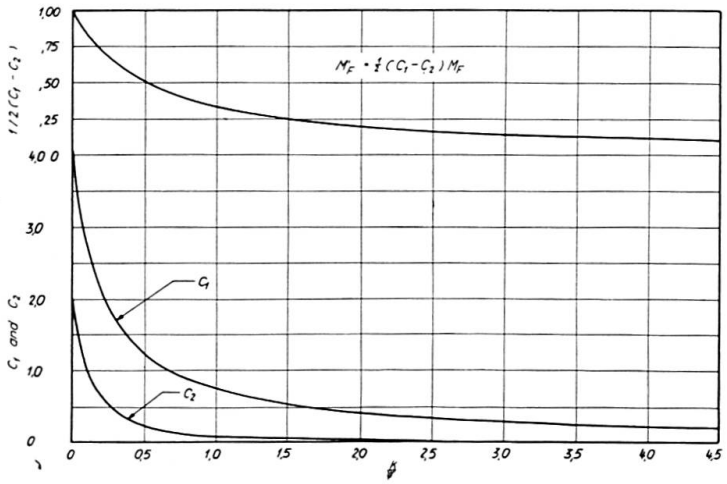

Fig. 5. Values of C_1, C_2 and M'_F.

of these diagrams is the rapid change that may occur in the values of C_1, C_2, and M'_F for small changes in K/Ψ. The changes in the fixed-end moments M'_F are indicated in Fig. 5 for a uniform load over the entire span. It is apparent that the fixed-end moments may change rapidly for even small changes in the K/Ψ values.

If a semi-rigid connection such as in specimen 3 (Fig. 3) is used instead of a rigid connection then a constant value of Ψ of 385×10^6 in-lbs is obtained from the slope of the M, ϕ curve. When this connection is used on a 12 WF 36 beam of 17 feet length the value of K/Ψ is

$$\frac{K}{\Psi} = \frac{E\,I}{l} = \frac{29 \cdot 10^6 \cdot 280.8}{17 \cdot 12 \cdot 385 \cdot 10^6} = 0.104\,.$$

From Fig. 5 or from Eqs. (5a, 5b, 5c) we obtain,

$$C_1 = 2.68\,, \qquad C_2 = 1.02\,,$$

$$M'_F = \tfrac{1}{2}(2.68 - 1.02)\,M_F = (0.83)\left(\frac{w\,L^2}{12}\right) = 0.069\,w\,L^2.$$

Summary

In this paper the importance of considering the deformation of beam connections in the design of steel frames has been emphasized. A laboratory procedure for determining the M, ϕ diagram for any type of beam connection has been discussed.

Analytical methods for incorporating the properties of the connections into the slope-deflection equations are presented. It has been shown that the stiffness of the beam and the magnitude of the end couples may be modified considerably by the rotational restraint factor Ψ of the connections. Therefore the actual beam coefficients and fixed-end moments, for the particular beam and connection, should be determined from test results and used in the structural analysis.

Résumé

L'auteur montre qu'il est important de tenir compte des déformations des attaches des traverses dans le calcul et l'étude des cadres métalliques. Pour des attaches quelconques, l'auteur présente une méthode qui permet de déterminer le diagramme de M en fonction de ϕ, au laboratoire.

L'auteur indique des méthodes analytiques permettant de tenir compte des propriétés des attaches dans les équations de la méthode des déformations. Il montre aussi que le facteur Ψ de l'attache (angle d'inclinaison de la courbe $M-\varphi$) influence fortement la rigidité et les moments d'encastrement total de la traverse. Les coefficients caractéristiques et les moments d'encastrement total de la traverse, nécessaires au calcul du système, devraient donc être déterminés par des essais pour la poutre considérée et le type d'attache utilisé.

Zusammenfassung

In diesem Beitrag wird die Bedeutung der Berücksichtigung der Nachgiebigkeit von Trägerverbindungen im Entwurf von Stahlrahmen betont. Für die Bestimmung des M-ϕ-Diagramms für jeden möglichen Trägeranschluß wird ein Laboratoriumsverfahren dargestellt.

Die Eigenschaften der Stöße werden dann analytisch in den Gleichungen der Deformationsmethode berücksichtigt. Es zeigt sich, daß der Einspannungsgrad die Trägersteifigkeit und die Größe der Endmomente wesentlich beeinflußt. Somit sollten die tatsächlichen Trägerbeiwerte und die Volleinspannmomente für einen besonderen Träger und seinem Anschluß aus einem Versuch bestimmt und dann in die Tragwerksberechnung eingesetzt werden.

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IIIa4

Beurteilung der Feuersicherheit von Stahlhochbauten

Evaluation of the Fire Resistance of Steel Structures

Appréciation de la sécurité des ouvrages métalliques contre l'incendie

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Dem Beamten der Feuerpolizei und dem Architekten muß ein einfaches, klares und übersichtliches *Punktsystem* in die Hand gegeben werden, aus welchem er sofort ersieht, ob die Stahlkonstruktion zu verkleiden ist oder nicht.

Das *Brandrisiko* setzt sich aus verschiedenen Faktoren zusammen: Zündquellen, Brennbarkeit, Feuerbelastung, Branddauer, Brandausbreitungsmöglichkeit, Verqualmung, Luftzutritt, Schadenanfälligkeit, Einsatz der Feuerwehr etc. — Die Feuerpolizei wehrt sich mit Recht dagegen, daß als einziges Kriterium für die Brandgefährdung die *Feuerbelastung* eingeführt wird. Dies war jedoch auch nie die Meinung, denn schon im Jahre 1950 ist durch E. Geilinger und C. F. Kollbrunner ein Punktsystem eingeführt worden, welches damals 15 Positionen umfaßte, wobei die Feuerbelastung lediglich als Position Nr. 7 aufgeführt wurde[1]).

Auch heute ist das Punktsystem noch nicht endgültig. Bis jetzt handelte es sich lediglich darum, die verschiedenen Meinungen zusammenzufassen und auf einen gemeinsamen Nenner zu bringen, damit darüber diskutiert werden kann. — Was wir wollen, ist ein *vereinfachtes Punktsystem*, welches in der Praxis ohne große Arbeit angewandt werden kann, ein Punktsystem, welches der neuzeitlichen Feuerbelastung Rechnung trägt. (Schlußendlich handelt es sich beim Punktsystem um Tabellen und graphische Darstellungen, die von E. Geilinger und C. F. Kollbrunner 1950 entworfen, durch P. Boué und

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[1]) E. Geilinger und C. F. Kollbrunner: Feuersicherheit der Stahlkonstruktionen, I. Teil. Mitteilungen der T.K.V.S.B., Heft 3. Leemann, Zürich, Mai 1950.

W. HALPAAP erweitert und heute als Diskussionsbasis aufgestellt wurden[2]).) Die Brandversuche werden in den meisten Ländern unter ähnlichen Bedingungen ausgeführt. Prinzipiell muß jedoch zwischen den Standardkurven und der Wirklichkeit ein klarer Trennungsstrich gezogen werden.

Für Ofenversuche, d. h. für die Beurteilung der Widerstandsfähigkeit von Verkleidungsmaterialien etc., muß streng nach einer Standardkurve vorgegangen werden, denn die verschiedenen Materialien müssen unter den genau gleichen Bedingungen untersucht werden, damit ein Vergleich möglich ist. — Ein Brand entwickelt sich jedoch nie nach einer Standardkurve, sondern je nach den örtlichen Verhältnissen und Gegebenheiten, bei viel Luftzutritt rasch, bei wenig Luft langsam, eventuell auch nur mottend.

Die absoluten Temperaturen beim Naturbrand liegen im allgemeinen viel tiefer als die Normkurve. Nur ganz kurze Zeit wird dieselbe überschritten. — Zudem wissen die Feuerwehrfachleute, daß es auch bei großer Feuerbelastung nur unter besonderen Verhältnissen möglich ist, über längere Zeit hinweg Temperaturen, wie sie die Standardkurve angibt, zu erzeugen.

Die Zeit-Temperatur-Kurve verläuft in der Praxis meist so, daß nach dem *Feuersprung* ein rascher Anstieg der Temperatur mit einer über der Standard-Kurve liegenden Spitze erfolgt, worauf ein flacher, langgezogener Abfall stattfindet. Dies liegt daran, daß die idealen Lüftungs- und Feuerungsverhältnisse der «Versuchsöfen» in den wenigsten Fällen in der Praxis vorhanden sind.

Fig. 1 zeigt die EMPA-Standard-Kurve für Ofenversuche und die Temperatur-Zeit-Kurven bei «natürlichem» Brand und langsamem Brand bei

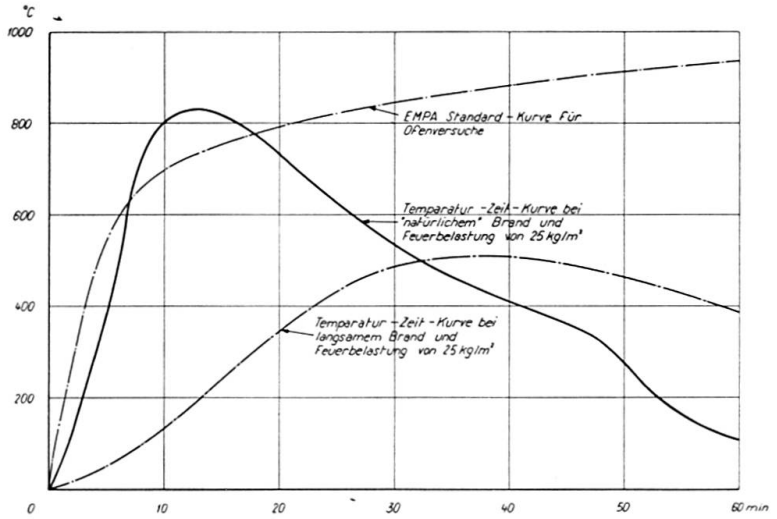

Fig. 1.

[2]) P. BOUÉ: Der Feuerschutz im Stahlhochbau, insbesondere von Stahlstützen. Berichte des Deutschen Ausschusses für Stahlbau, Heft 21. Stahlbau-Verlags GmbH., Köln, 1959.

W. HALPAAP: Die Bestimmung des notwendigen Schutzes wesentlicher Bauteile nach Punkten. VFDB-Zeitschrift, Heft 4, S. 124, November 1959.

Feuerbelastungen von 25 kg/m². Bei mottendem Brand verläuft die Kurve noch viel flacher.

Betreffend die neuesten publizierten Punktsysteme wird auf die Fußnote verwiesen [3]). Aus der Tabelle ersieht man die Klassifikation der Stahlhochbauten, wobei hier eindeutig angegeben ist, daß bis zu einer Feuerbelastung von 25 kg/m² die Punktzahl keine Rolle spielt.

Tabelle. Klassifikation der Stahlhochbauten

Gefahrenklasse	Feuerbelastung kg/m²	Punktzahl (minimal)	Stahlkonstruktion	Art der Gebäude
1	bis 25	spielt keine Rolle	unverkleidet	Wohnhäuser, Geschäfts- u. Bureauhäuser, kleine Hotels und Schulhäuser, Hallen
2	über 25 bis 50	über 15	unverkleidet, teilw. leicht verkleidet (Kernfüllung)	mittlere Verkaufslokale, größere Hotels und Schulhäuser, Lagerräume mit nicht sehr viel brennbarem Material
3	über 50 bis 100	über 25	leicht verkleidet	große Verkaufslokale, Warenhäuser, Spitäler, Lagerräume großer Geschäftshäuser
4	über 100 bis 150	über 35	stark verkleidet	große Hotels, Theater, Kinos, große Versammlungslokale, Großgaragen, Lagerhäuser mit viel brennbarem Material oder leicht brennbaren Stoffen

Festgehalten werden muß, daß es grundsätzlich falsch ist, nur nach Punkten zu bewerten. (Extremfall: *Viele* Punkte, jedoch *keine* Feuerbelastung.)

In den Feuerpolizeivorschriften gelten meist als «feuerbeständig» Bauteile aus nicht brennbaren Baustoffen, die während mindestens 90 Minuten unter der Einwirkung des Feuers und des Löschwassers ihr Gefüge nicht wesentlich ändern und damit ihre Tragfähigkeit und ihre Standsicherheit nicht über das zulässige Maß verlieren. — Schon ein Laie erkennt, daß es unlogisch ist, bei modernen Bureau- und Geschäftshäusern, mit einer Feuerbelastung von 8 bis maximal 25 kg/m², d. h. einer für den Einzelraum maximalen Branddauer von ca. 20 Minuten (sofern man als Branddauer die Zeit des Erreichens und

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[3]) C. F. Kollbrunner: Bewertung des Feuerschutzes des Stahlkonstruktionen nach dem Punktsystem. Schweiz. Bauzeitung, Heft 9, S. 142, 3. März 1960.

P. Boué: Beitrag zur Frage des Feuerschutzes von Stahlhochbauten. I.V.B.H., sechster Kongreß, Stockholm 1960, Vorbericht, S. 421.

teilweise Überschreitens der maximalen Temperaturen in der Höhe der Standardkurve annimmt), eine Brandsicherheit für 90 Minuten zu verlangen. Eine so lange Branddauer kann in modernen Bureau- und Geschäftshäusern überhaupt nicht mehr auftreten.

Fig. 2 zeigt ein vereinfachtes Punktsystem. (Einfache graphische Darstellung, wobei über die Größe der + - und – - Punkte noch verhandelt werden kann.)

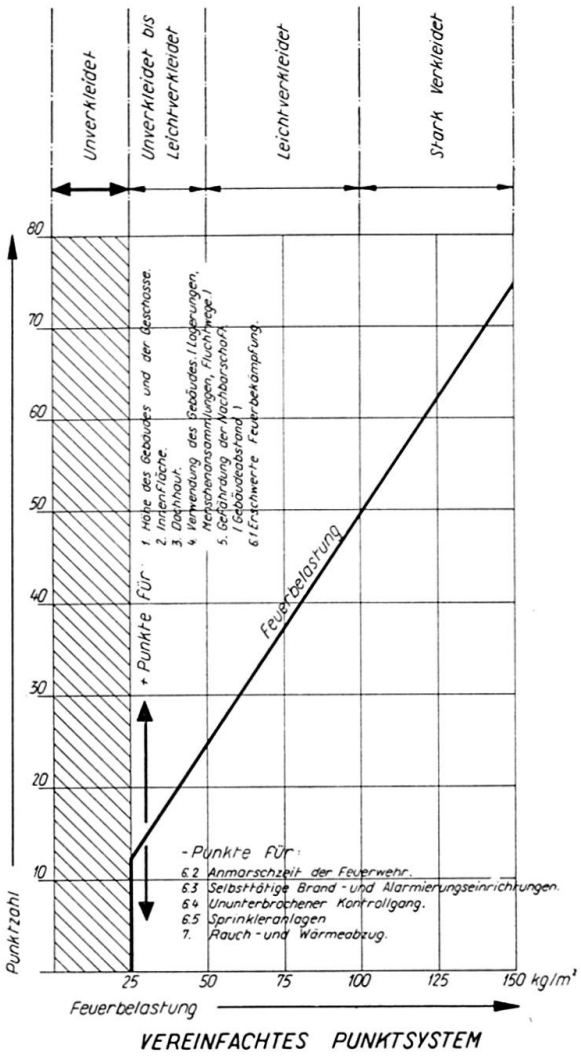

Fig. 2.

Wichtig ist, daß aus diesem vereinfachten Punktsystem abgelesen werden kann, daß bis zu 25 kg/m² Feuerbelastung die Stahlkonstruktionen *unverkleidet* ausgeführt werden können. (Alle Stahlkonstruktionen, Außenstützen, Innenstützen etc.)

Wir stehen heute vor der Verwirklichung eines einfachen Punktsystems. In Zukunft müssen viel weniger Stahlkonstruktionen verkleidet werden als bisher.

Zusammenfassung

Ergänzungen zu den Artikeln III a 1 (PAUL BOUÉ) und III a 3 (C. F. KOLLBRUNNER) des «Vorberichtes».

Bewertung der Stahlhochbauten nach einem vereinfachten, für die Praxis entwickelten Punktsystem, welches gestattet, sofort anzugeben, ob die Stahlkonstruktion unverkleidet ausgeführt oder verkleidet werden muß.

Summary

Complements to the contributions III a 1 (PAUL BOUÉ) and III a 3 (C. F. KOLLBRUNNER) in the "Preliminary Publication".

Evaluation of steel structures according to a simplified point method developed for practical use, indicating immediately whether the steel construction can be executed uncased or if casing should be provided.

Résumé

Compléments à l'article III a 1 (PAUL BOUÉ) et III a 3 (C. F. KOLLBRUNNER) de la «Publication Préliminaire».

Appréciation des ouvrages métalliques d'après un système simplifié développé pour la pratique et permettant d'indiquer immédiatement si la construction métallique doit être revêtue ou non.

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IIIa5

Discussion on theme IIIa[1])

Remarques concernant le thème IIIa[1])

Diskussionsbeitrag zu Thema IIIa[1])

DONOVAN H. LEE
London

The paper by Mr. PICKWORTH on American practice with steel framed tier buildings is a valuable summary of current practice there, especially taken in conjunction with the paper by Mr. STETINA covering American practice with floor constructions. Possibly however, the remark that the saving of weight in high buildings is important for economy although true can be misleading as the cost of carrying extra weight even for fairly high buildings is seldom sufficient to pay for much increase in the cost of a floor construction.

The writer has shown the cost of carrying weight in the form of a chart, which can be prepared for any particular design stresses for columns and beams, on the assumption that the designer will work closely to the permissible stresses, and the extra cost of the steel per ton of extra weight carried can be read directly in cost per ton. On the whole, comparisons in this way tend to show that American practice is based to a large extent on speed of construction. It is true of course that long span floors of light construction increase the need for other ways to obtain the necessary resistance to the transmission of sound, but the general use of false ceilings helps. It seems important to reduce the reflection of sound in offices and the widespread use of acoustic tile ceilings in America will, the writer suggests, have to be copied more in the United Kingdom, where generally speaking the building owners are not easily persuaded to incur the extra cost.

With regard to the paper by SPARKES, CHAPMAN and CASSEL, the valuable data on stresses measured in this particular steel framed structure is an excellent supplement to the stresses found by the British Building Research

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[1]) See "Preliminary Publication" — voir «Publication Préliminaire» — siehe «Vorbericht», p. 433, 467, 479, 493.

Station with the Government offices in Whitehall a few years back. In both cases the stresses found are on the whole lower than would have been expected by ordinary calculations, particularly so as regards the live load. However, it would be of interest if the authors could explain the comparatively large initial stress mentioned of 3 tons per square inch caused in erection and by welding.

Referring to the paper by Messrs. WRIGHT and GOODERHAM and in particular to the various types of welded connections used in recent tier building frames in Canada, there would seem much to favour the type of detail shown by Fig. 5, not only for the advantages mentioned by the authors on page 499, but also because excessively large negative moments can be prevented, which might for example be due to relative settlement of foundations or to increased stiffness of some parts of the structure by composite action or even by relative axial strains in columns. The writer dealt briefly with this matter from the point of view of safety in connections in "Engineering News Record" May 14th 1959. Nevertheless on foundations in which relative settlement is not to be expected as in the case of the tower frame shown by Fig. 7 on page 501, such objections might not be of much consequence.

The writer would like to know if the authors agree that the ideal steel structure would be one in which a beam can be lowered into position and be immediately safe against being hit accidentally. It is assumed that the long web cleats shown on Fig. 8 is the alternative needed in that case because of desire to avoid a landing cleat and a safety clip near the top of the beam connection which would have only needed one bolt.

Summary

The author gives some remarks on the papers IIIa 2, 5, 6, 7. (Prelim. Publ.)

Résumé

L'auteur fait quelques remarques concernant les contributions IIIa 2, 5, 6, 7. (Publ. Prél.)

Zusammenfassung

Dieser kurze Diskussionsbeitrag enthält Bemerkungen zu den Arbeiten IIIa 2, 5, 6, 7. (Vorbericht.)

IIIa6

Discussion - Discussion - Diskussion

Two Problems Relating to the Design of Framed Tier Buildings (Pierre Dubas, IIIa2) [1])

Deux problèmes relatifs à l'étude des portiques étagés multiples (Pierre Dubas, IIIa2) [1])

Zwei theoretische Untersuchungen an mehrstieligen Stockwerkrahmen (Pierre Dubas, IIIa2) [1])

DONOVAN H. LEE
London

The author has reminded us of the effect of change of length due to axial forces on the moments in the members of a steel framed tier building; in this case the lower 6 storeys of a future 26 storey structure.

While wind moments are one cause of differential axial strain in columns, variation in superimposed load is another and generally affects internal columns more than external. For steel frames with conventional external walls it seems unlikely the full axial strains occur in columns due to wind; the walls take some, even most, of the compression. With curtain walling however, this is not to be expected.

As the author mentions, the effect on moments in external columns due to axial strain is greatest at the top when caused by wind force, however in the case of internal columns moment due to incidence of the superimposed loading can be greatest at the top and will never be the least there.

The columns of the authors example would shorten under full load about 30 mm so with no load on the floors the internal columns might shorten only some fraction of that. The writer has never seen cracks from either cause of axial strain, and it seems reasonable to believe the floor constructions tend to

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[1]) See "Preliminary Publication" — voir «Publication Préliminaire» — siehe «Vorbericht», p. 433.

equalise the column strains, so that the authors moment diagrams Figs. 2 and 3 will be modified for cases of stiff floors.

Of course, variable settlement of foundations could have more effect on the moments than the strain due to either wind or incidence of loading, but it is assumed the building referred to is founded on rock.

Summary

The author reminds us of the effect in steel framed tier buildings of conventional walls, curtain walls, stiff floors and foundation-settlements.

Résumé

Pour les portiques étagés multiples, l'auteur rappelle l'influence des parois traditionnelles, des murs-rideaux, des dalles rigides et des tassements des fondations.

Zusammenfassung

Der Autor erinnert an die Auswirkung in Stahlstockwerkrahmen von konventionellen Wänden, Vorhangwänden, steifen Decken und Fundamentssetzungen.