General report

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General Report

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The diversity of contributions to the Theme of Steel Framed Buildings is a reflection of the diversity of engineering requirements and skills which must combine to produce efficient structures of this type. Among this diversity, the structural design is only one facet. It is fitting that at least some of the other facets are reflected in the discussion, such as: the functional and economic (in contrast to structural) features of wall, floor, and roof construction, the manner in which erection procedures affect the economy of the structure, the problem of fire proofing, and others.

I. Structural Design

In the Preliminary General Report it has been emphasized in some detail that the question of frame instability (side-sway buckling) is becoming increasingly important. This is so because of two simultaneous tendencies. On the one hand, the reduction in dead loads achieved with modern wall and floor construction, as well as improved methods of design, analysis, and construction have resulted in much lighter steel frames than were current, say, in the 1930. The resulting greater slenderness of the columns makes buckling phenomena more important than before. It can be predicted that this tendency will be further accentuated by the forthcoming use of higher strength steels for ordinary construction. This development is just now making its appearance in the U.S.A. with the recent introduction of several new structural steels covering a considerable range of strengths. On the other hand, the present-day light curtain walls and partitions contribute little, if anything, toward the lateral stability of the structure, in contrast to the much greater rigidity of the older types of heavy walls. It is evident that the simultaneous development toward more slender columns and toward reduced wall bracing greatly accentuates the problem of lateral buckling.

Prof. GOLDBERG's discussion represents a further contribution to this topic. Its urgency is illustrated by the following significant quotation from that contribution: "Calculations made upon some recently designed building frames for the sidesway mode of buckling show that the equivalent or effective column length may be as much as three story heights. This is far from the one-story assumption which, in the past, has been a convenient and apparently adequate basis of design." This information is consistent with that provided by W. MERCHANT and A. H. SALEM in the Preliminary Publication, and described in the General Report. It represents a definite warning to the effect that conventional methods of column design, which used to disregard the possibility of lateral displacement, must be supplanted by more refined and realistic methods of analysis, or else decidedly unsafe situations may result.

The substance of Prof. GOLDBERG's contribution is concerned with structures which are braced against side-sway not by the stiffness of the frame itself, but by shear walls, truss bracing, or other special vertical bracing systems. Methods are shown for calculating the minimum stiffness so that such bracing systems will prevent side-sway buckling. Once such lateral instability is prevented, the only remaining buckling mode is that in which the two ends of each column do not move laterally with respect to one another; this mode results in much larger buckling loads than the side-sway mode.

In contemplating the design significance of Mr. GOLDBERG's contribution, two observations should be made which are both based on a recent paper by the present reporter on similar problems of lateral bracing [1][1]). In that paper

[1]) References p. 211.

the type of bracing required to prevent side-sway buckling, and analyzed by Mr. GOLDBERG, has been called "full bracing". The rigidity required for full bracing of an ideally aligned frame subject to vertical loads only can be designated by k_{id}. It is this value which is analyzed by Mr. GOLDBERG. On the other hand, any real member or frame is affected by inevitable defects of alignment, straightness, etc., and by the further possibility of transverse loads from wind and other sources. It is shown in [1] that the actual rigidity k_{req} which is required to prevent sidesway buckling in the presence of these inevitable factors, is larger than k_{id} and can be computed if these factors are known or reasonably assumed. It follows from this that Mr. GOLDBERG's criteria actually represent lower limits for the required shear rigidity of the bracing systems and that for actual structures larger rigidities must be provided.

On the other hand, it should not be forgotten that it is the primary function of these bracing systems to resist definite horizontal forces, chiefly wind loads, and it is for these loads that such bracing is chiefly designed. It seems likely for most buildings of significant height that a bracing system adequate to resist wind and other horizontal forces will be more than adequate to resist side-sway buckling, provided that the individual frames are connected to the shear walls by the customary, rigid floor diaphragms. However, should there be a case where the actual rigidity of the bracing system is not greatly in excess of the value k_{id} obtained from Mr. GOLDBERG's and similar calculations, then it becomes necessary to enter upon a second order analysis for determining that actual rigidity which is required to prevent side-sway failure due to the combined action of the buckling tendency from vertical loads plus the lateral deflections produced by wind and other horizontal loads. It will then be found that k_{req} for these actual conditions exceeds k_{id} by very considerable amounts.

Mr. H. BECK elaborates on Mr. P. DUBAS' investigation of the effect of axial shortening on moments in tier building frames. He analyzes a cantilever Vierendeel frame as representing the action of a building frame under wind load and shows that the influence of axial deformation increases with increasing ratio of girder stiffness to column stiffness and with the number of stories. Since the influence is particularly significant for slender columns one wonders whether another factor, mostly neglected in design, would not be found of equal or greater consequence. This is the loss in effective rigidity of the columns caused by their axial loads, and the corresponding increase of wind deflections and moments over those calculated by neglecting this factor.

Mr. BÖHMER deals with a problem of considerable practical interest, namely with the strength of a column which has been reinforced by means of added plates of a steel of higher strength than that of the original member. The problem, however, is not restricted to this situation. With the above mentioned wider availability of structural steels of a variety of strengths, structures are now being erected where several steels are used in combination, depending on the functions of the particular member or element. Evidently, for a member where rigidity is the primary requirement, such as in columns of considerable slenderness, an inexpensive low-strength steel is the most economical material. On the other hand, where yield strength is the primary requirement, higher strength steels are appropriate. Along this line of thought, long-span plate girders for bridges have recently been built where, within the same main girder, a variety of steels have been used, i. e. lower strength steels in the lowly stressed portions near inflection points, and higher strength steels, particularly for the flanges, in high-moment regions. By analogy it is possible that Mr. BÖHMER's idea of obtaining a higher strength column by reinforcing I or WF section with high strength flange plates could prove to be economical not only for subsequent strengthening of existing structures, but also in the original design of new structures, particularly if the columns are also subject to bending moments.

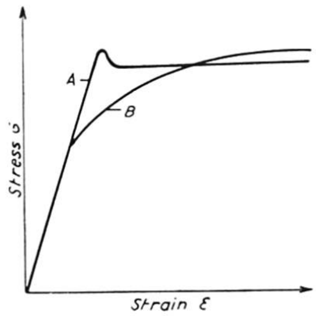

Fig. 1. Types of stress-strain diagrams.

A: Sharp-yielding (annealed steel)
B: Gradual-yielding (with residual stresses)

Mr. BÖHMER's contribution is a convincing example of a situation where elastic calculations would be entirely misleading, and where a rational and realistic analysis can be achieved only by including the effects of the plastic properties of steel. The writer would merely like to add the following observations to Mr. BÖHMER's ingenious method. It is assumed in this method that the column strength curve σ_{cr} vs. λ is of a type determinded by presumptive initial curvature, such as in DUTHEIL's method, among several others. From this it would follow that strengthening with higher strength steel would change the ordinates, but not the shape of this column curve. However, it has been shown in recent years by very extensive research [2], [3], [4], [5], [6] that the shape of column curves in the range of low and medium slendernesses depends not on fictitious or presumptive initial eccentricities, but on the influence of very real and measurable residual stresses. In rolled shapes these are caused by the cooling process, in welded members by the residual temperature stresses, and in cold-formed members by the localized effects of cold working. The main effect of these factors is to change the effective stress-strain curve of the material in the member from the sharp yielding character of annealed steel (Fig. 1, curve A) to the gradual yielding type of curve B. The corresponding reduction of the effective modulus is the reason for the departure of the column curve from the elastic Euler curve in the region of low and medium slendernesses. THÜRLIMANN [2], from tests carried out at Lehigh University, reports the following ratios for σ_{cr}/σ_y for columns with slenderness ratio $\lambda = 90$, made of standard mild structural steel: annealed WF shapes -0.9, riveted I-section -0.85, as-rolled WF shape -0.75, welded I-section -0.60. This is a dramatic illustration of the influence of the magnitude of residual stresses and cannot be explained by any of the column theories based on presumed initial curvature or eccentricity. Now, if the strengthening process envisaged by Mr. BÖHMER is carried out by welding, it is practically certain that the resulting cooling stresses will affect the effective stress-strain curve of both materials, the I-shaped core as well as the reinforcing plates in a manner similar to the quoted figures. This remark by no means invalidates the basic concept of Mr. BÖHMER's analysis. It only emphasizes an additional factor which may significantly affect the real buckling strength of such reinforced columns.

In addition to simple connections whose moment resistance is negligible, and to fully rigid connections which are utilized where complete continuity, elastic or plastic, is desired, semi-rigid connections have at times been intensively investigated. They do not provide full continuity, but do develop moments which are sufficiently large to make their utilization economically attractive. This method of design has not found wide application, primarily for two reasons: On the one hand, each type and size of connection has its own moment-rotation characteristics which must be known in order to be incorporated in design; this requires a large amount of experimental information to be available. On the other hand, the complications in analysis produced by partial continuity are quite considerable and time consuming. Prof. MAUGH proposes a simple test method for obtaining the moment-rotation relation of any particular connection for use in the given design. He then linearizes the strongly non-linear relationship and incorporates it in a modification of the slope deflection method of analysis. It might be desirable to establish by full scale frame testing whether the analysis so obtained gives results which are adequately realistic.

II. Erection

Mr. SCHMID describes French developments toward lightweight, prefabricated floors and curtain walls similar to, but not entirely identical with those in the U.S.A., which were discussed in the Preliminary General Report and described in detail in the Preliminary Publication by Mr. KRAPFENBAUER and Mr. STETINA. Mr. SCHMID's process of casting a stack of concrete floor slabs on the ground and then lifting them into place is similar to the "lift slab" procedure which has been long and successfully used in the United States. Differences, however, exist. In the American procedure slabs of much larger area, often covering the entire floor area of the building and supported on any number of columns, are cast and then lifted in one piece. This requires free standing columns and high precision in the lifting process, which is carried out by the simultaneous action of a number of jacks, one each for every column. As a consequence, this method has been found practical mostly for structures only a few stories high. In contrast, in Mr. SCHMID's procedure the individual slabs are of smaller size, each supported only by four columns at the corners. This makes possible the relatively simple lifting and mounting procedure described by the contributor, and apparently enables this type of construction to be used for buildings as high as twelve stories. It might be added that under favorable conditions the American lift slab method has been used for apartment buildings up to 12 and 14 stories high. Each concrete slab covered the entire floor area as one monolithic unit, 215×69 ft. in the latest structure of this kind in Ann Arbor, Michigan [7], and was lifted by 36 electronically controlled, simultaneously acting jacks.

The assembly of entire multi-story frames on the ground and their final placement by tipping them up as described by Mr. SCHMID, is certainly unusual and requires skillful erection procedures and careful checking of erection stresses. While space limitations will generally prevent this method from being used in crowded city conditions, it seems to have considerable possibilities where no such restrictions exist. In the erection of the initial bents, the single mast which alternatively serves as boom and as pole has certain features in common with the jumping guy derrick described by Mr. RAPP and illustrated in his splendid photographs. The latter is probably simpler in operation and capable of greater precision, which is appropriate since the entire erection is carried out by means of this derrick. Mr. SCHMID's single boom, on the other hand, is used only for mounting the initial bents, and in view of this modest task this simpler, but somewhat more cumbersome tool is probably economically appropriate.

The problems and methods of wind bracing described by Mr. SCHMID also parallel those discussed in the Preliminary Publication. Clearly, regardless of country, the same problems arise and similar solutions present themselves for structures of similar type.

III. Fire Protection

Conceptually, the problem of fire protection has much in common with the problem of structural safety. This similarity becomes very apparent when one examines the contributions to this theme by Dr. KOLLBRUNNER and Mr. BOUÉ in the Preliminary and the Final Publication.

Both in the field of structural safety and in that of fire protection, there exists certain objective information of a basic nature. Thus if a structure is made precisely to the design drawings, and of materials of precisely known properties, and is acted upon precisely by the design loads then, at least in simpler cases, it is possible to calculate with considerable precision the actual strength of the structure and to determine the so-called safety factor inherent in the design. Likewise, from numerous fire tests and other information, for a structure with precisely known fire resistance, precisely known combustible content, and precisely known time-temperature curve, the fire performance becomes highly predictable and the degree of fire safety can be expressed in definite terms.

On the other hand, in regard to structural safety, a large number of decisive factors are of an uncertain and, at best, statistical nature. Materials properties vary over considerable ranges, actual loads are different in magnitude and even in nature from assumed design loads, humanly inevitable construction inaccuracies affect internal stresses, design calculations are only approximate since they are based on certain idealizing assumptions, and consequences of structural failures, in terms of loss of life and property, vary over a wide range. Some of these influences can be evaluated statistically and their results incorporated in a probabilistic concept of structural safety. For others, engineering judgement must be used because either the necessary data for statistical treatment are not available or because the factors themselves are of a non-statistical nature. — In a similar manner, in regard to fire safety, analogous factors exist which are also of an uncertain and, at best, statistical nature. As Mr. KOLLBRUNNER points out, in actual fires the time-temperature curves vary widely and differ from those standardized in fire tests; actual combustible contents also vary and only statistical data, if any, are available; the actual efficacy of fire protection measures depends on a number of random influences which may not be reflected in fire tests; and the consequences of fire damage vary over a similarly wide range as those of structural failures.

It would seem, therefore, that a probabilistic approach, tempered by engineering judgement applied to the non-statistical features of the problem, is equally appropriate in the field of fire safety as it is in the field of structural safety. It is no accident that the point system of evaluating fire protection, as proposed in the contributions by Mr. BOUÉ and Mr. KOLLBRUNNER, is somewhat analogous to the rating system for structural safety advanced by the Institution of Structural Engineers in London [8]. It is the undoubted advantage of such point or rating systems that they reduce the influence of subjectivity and tend to produce greater uniformity. They result in definite numbers which characterize the given situation. One should not forget, however, that these numbers do not have the same degree of validity as have objectively measurable quantities, such as stresses, deformations, etc. Nor do they have the same probabilistic significance as have quantities of known statistical distribution, such as certain materials properties, load intensities, etc. Point or rating systems are basically numerical codifications of engineering judgement based on so-called common sense. When this limitation is kept in mind, and when they are not regarded as objective truth, they are of undoubted advantage and represent valuable developments toward more realistic and uniform procedures.

IV. Conclusions and Recommendations

The economic and social conditions which promote the need for high multi-story tier buildings are the same in all industrial countries. They have developed somewhat earlier in the U.S.A. where the tall building, from 15 to 40 stories and more, has been the normal metropolitan structure for decades, but similar developments are now found almost everywhere. At the same time this structural type, since World War II, has undergone a minor revolution in functional and structural aspects; again with some differences in speed of development, the features of this revolution are similar in the various countries.

For one thing, the functional nature of these structures has changed and continues to change. While formerly their primary function was that of shelters, and their shape and layout were practically permanent throughout the life of the structure, they are now becoming increasingly complex machines, highly adapted to their individual purposes and at the same time endowed with maximum internal flexibility and adaptability.

Structurally, the demand for economy and speed of erection has resulted in the preponderant use of prefabrication, not only of the elements of structural framing, but also of floors, walls, partitions, and other units. In the course of these developments, lightweight construction has replaced the traditional, heavy installations, with corresponding savings in time, and in cost of framing and foundations. New connection methods, welding and high strength bolting, have all but eliminated riveting. New approaches to fire protection have further intensified the trend toward weight reduction. Demands for internal flexibility have led to large column spacings and often to complete elimination of interior columns.

Reduction in dead load, lightening of the frame, and lack of rigidity of flexible curtain walls and partitions are imparting decisive importance to the manner in which such structures are made to resist horizontal loads from wind, earthquake and other causes. In regard to stability, the side-sway mode of buckling, which used to be disregarded in conventional column design, takes on increasing importance. Other new departures in methods of design and analysis place increasing emphasis on actual strength and actual structural performance under load, as distinct from formal, purely elastic stress analysis.

Many of these developments were possible only on the basis of extensive and systematic research efforts on a large scale. This refers to the behaviour of welded and of high strength bolted connections, to the development of thin-walled, coldformed steel members, to research on frame stability and on the ultimate strength of steel structures (often somewhat mistakenly designated as "plastic design"), to methods and effects of fire protection, and others. In this respect, two recommendations come to mind:

1. With the radical reduction in dead loads, the influence of live loads, both vertical and horizontal, becomes of increasing importance. The degree of safety of a structure is intimately related to the degree of certainty with which the loads are known. It would seem that a concerted effort should be made to investigate, by actual measurement and statistical evaluation, the intensity and distribution of live loads, including wind loads, which act on tall, multi-story structures. The large efforts which have been expended in research on stress, strain, and strength, to be made fully effective, must be supplemented by corresponding investigations on conditions of loading.

2. The immediate effects of the above mentioned, large-scale research undertakings have all too often been limited to the countries where the research was done. This has retarded development and led to superfluous duplication of effort. It is realized that traditions as well as specific conditions often make it impossible to transfer without change technical developments of one country to another. Yet, more complete knowledge of such developments is advantageous to all. Thus, the U.S.A. has greatly profited from a systematic study of European developments in prestressed concrete prior to embarking on its own evolution in this field. All too often, however, such cross fertilization is absent.

Here is an area in which the I.A.B.S.E. could provide increased service. This would consist in the systematic publication of summary articles, that is, of papers which review and summarize in a systematic manner new developments in a specific country and field. Precedents for this are the papers by Mr. Stetina and by Mr. Rapp which were written on the specific request of President Stüssi. Other examples are those of [2] and [5]. In this manner the Congresses as well as the annual Publications could be of increased assistance to the openminded and unprejudiced reader who wishes to utilize new technological developments regardless of national origin.

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Rapport général

La diversité des contributions au thème III, concernant les ossatures métalliques, reflète bien la diversité des exigences et des connaissances techniques dont seule la combinaison permet de réaliser des ossatures satisfaisantes. De cette diversité, l'étude statique n'est que l'un des aspects. Il convient que quelques autres aspects au moins du problème apparaissent dans la discussion, par exemple les particularités fonctionnelles et économiques (par opposition au point de vue statique) des parois, planchers et toitures, l'influence des procédés de montage sur le coût d'un ouvrage, le problème de la protection contre l'incendie et d'autres encore.

I. Etude et calcul des ossatures

Dans la «Publication Préliminaire», j'ai souligné à diverses reprises l'importance croissante du problème de l'instabilité des portiques (flambage par déplacement latéral). Deux tendances simultanées sont ici en cause. La diminution du poids mort, due à l'utilisation de parois et de planchers modernes, ainsi que des méthodes améliorées d'étude, de calcul et de construction, d'une part, permettent de réaliser des ossatures métalliques bien plus légères que celles courantes vers 1930, par exemple. Il en résulte un plus grand élancement des poteaux, ce qui accroît l'importance des phénomènes de flambage. Il est à prévoir que cette tendance s'accentuera lorsque l'on utilisera de l'acier à haute résistance pour des ossatures même peu importantes. Aux Etats-Unis, cette évolution vient de se manifester avec la récente introduction de plusieurs nouvelles nuances d'acier, couvrant un important domaine de résistance. Les cloisons et murs-rideaux légers actuels, d'autre part, comparés aux anciens types de murs lourds très rigides, ne contribuent que peu, ou pas du tout, à augmenter la stabilité latérale de l'ossature. Cette évolution simultanée vers des poteaux plus élancés et une diminution de l'effet de contreventement des parois ne peut qu'augmenter considérablement l'importance du problème du flambement latéral.