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General Report

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1. Current Trends in Steel Building Design

The major part of this report is concerned with steel-framed multistory buildings of the type used for offices, hotels, apartments, and the like. While many important refinements and improvements are taking place in the design methods for such structures, no radical change has yet occurred in this field. Yet, in 1960 no report on steel-framed buildings would be complete without mention of the fact that a new and basically different design method is establishing itself, at least for one- and two-story welded rigid frame structures, such as industrial buildings, warehouses, and the like.

I am referring, of course to design and analysis based on the plastic behavior of structural steel. Various phases of this method have been discussed in 1956 in Lisbon and, particularly, in 1952 in Cambridge. Since that time the codes and specifications for the design of steel structures, both in Great Britain and in the United States, have officially authorized the use of these methods. This was the result of intensive and sustained research efforts carried out for more than a dozen years semi-independently in Great Britain (mostly at Cambridge University) and in the United States (mostly at Lehigh and Brown Universities). In the USA, in particular, the results of this research have been incorporated in an appendix to the nationally recognized Specification of the American Institute of Steel Construction. This appendix, adopted in December 1958, authorizes the use of the plastic design method and contains specific rules for its application. It limits the application to continuous beams and one- and two-story rigid frames. Individual structures, both in Britain and the USA, had been plastically designed even before the official adoption of these rules. Their number is now increasing significantly, even though both methods continue to exist side by side. What recommends the plastic method, if judiciously applied, is a demonstrable saving in steel, a reduction of the time required for design and analysis, and a more realistic understanding of the actual behavior of continuous steel structures under load.

Even though further research must be done, particularly in the field of plastic frame instability, to show whether plastic design can be applied with assurance to high, multi-story buildings, its lessons even now deepen our engineering understanding of such structures. The interesting contribution of Mr. Pierre Dubas may serve as an illustration. By an elegant elastic analysis Mr. Dubas demonstrates the influence of axial deformations of the columns on the distribution of bending moments in multi-story frames. He shows, in particular, that even if the loads on the frames are concentrically applied directly to the columns, sizable moments are induced merely by the differences in longitudinal column deformations. This result is interesting and undoubtedly correct. However, plastic theory leads one to believe that these moments, merely caused by elastic deformation compatibility, have little influence on the real carrying capacity of the frame for static loading. Indeed, partial plastification prior to collapse tends to cancel these deformation-caused moments and to render them harmless. The case is somewhat analogous to that of differential support settlement of limited amount. This, too, looks very serious when only elastic stresses are investigated, but is actually harmless to the strength of the structure, as is easily demonstrated by plastic analysis and test.

One of the reasons why caution is still advisable in applying plastic analysis to multi-story frames is well illustrated by the results of the important investigations on frame instability carried out at the College of Science and Engineering, Manchester, England, and briefly summarized in the contribution by W. Merchant and A. H. Salem. They demonstrate that the loads at which multi-story frames are subject to side-sway buckling (i. e. buckling by horizontal motion even though all external loads may be vertical) are considerably smaller (up to 50% and more) than one would obtain by considering the columns as simple, hinge-ended Euler columns. This is increasingly so, the larger the number of stories. It follows that for the correct elastic design of such frames effective lengths for the columns must be used which are larger than the real length. With regard to plastic frame analysis the paper further demonstrates the following by specific examples: For a simple portal frame under chiefly vertical loading, frame instability may cause the actual collapse load to be only 80% of that indicated by «simple plastic theory». For two-story portals, frame instability can reduce the collapse load by as much as 30% as compared to that which would be plastically calculated without regard to instability, for slenderness ratios of the columns as low as 67. The A.I.S.C. rules for plastic design make provision for such side-sway buckling, but these are so far not applicable to tall, multi-story frameworks.

This matter of frame instability by lateral buckling is gaining increasing importance, regardless of whether frames are designed elastically or plastically. This is so because, as will be discussed below, modern construction tends to decrease dead loads radically. In consequence, we now have much lighter frames than were customary as recently as ten years ago. This tendency also heightens the influence of horizontal loads, such as by wind or seismic action, which further accentuates the problem of the lateral stability of such frames.

These two features, plastic design and lateral frame stability, have been mentioned first because, in their application to steel framed tier buildings, they are in a state of flux and of continuing research. The rest of this report will be concerned with a description of the present status and techniques in the construction of tier buildings, in which radical changes have occurred within the last ten years. In addition to the papers in the Preliminary Publication, reference will be made chiefly to American developments. The writer wishes to apologize for this one-sided treatment, but he is necessarily limited by his own experience.

2. Nature and Function of the Modern Tier Building

It is impossible to discuss adequately the structural aspects of modern tier buildings without considering their functional nature. As Mr. Pickworth has pointed out in his contribution, the modern office building is a complicated machine; it must be viewed as such rather than merely as a structure. The proper functioning of modern office organizations, banks, and the like requires a multitude of services, among which are internal and external telephones and closed-circuit television, water, gas and sanitary services, electric power for a wide variety of uses, vertical and horizontal traffic within the building, heating and air conditioning, acoustical conditioning, and provision for the rapidly expanding use of a great variety of business machines. To avoid premature obsolescence, all these services must be kept as flexible and adaptable as possible. In addition, it is these high buildings which increasingly determine the face and profile of the modern city, so that architectural and esthetic demands assume primary importance. The president of one of the large construction companies, Mr. H. C. Turner, has said that in regard to modern tier buildings "contemporary design means a desire for distinction and individuality, for greater efficiency, usability of space, flexibility, and expansibility." The attainment of these goals depends on a close co-operation of a team of architects, mechanical, electrical, sanitary and other engineers and, of course, of structural engineers. As Mr. Pickworth remarks, the building will be successful if the structural engineers enter into the very earliest phases of planning, rather than being called upon merely to furnish the structural design for an architecturally completed project.

From a technological viewpoint the modern tier building is increasingly becoming an assembly of factory mass-produced parts shipped to the site and hoisted into place. Cellular and other floor units, movable interior partitions, flat ceilings of acoustic tile or translucent plastic, recessed fluorescent lighting, thin exterior curtain walls, and sub-assemblies of the previously mentioned service conduits, are all mass prefabrications. They reduce greatly the amount of on-site labor and eliminate much, if not all, of exterior and interior scaffolding and shoring. All this contributes to speed and economy of erection, particularly under the difficult construction conditions in the center of big and crowded cities.

The demand for complete flexibility of use of space has led to one of the more radical structural developments, namely to a constantly increasing spacing of columns. During the last few years several high office buildings have been erected in the USA and Canada which are entirely free of interior columns. In these particular structures the two rows of exterior columns are spaced approximately 60 ft. from each other and the floor system is supported by girders approximately 3 ft. deep which span freely across the 60 ft. between columns. This is an illustration of the fact that in such buildings functional demands frequently outweigh possible economies in the initial cost of the structure, as pointed out by Mr. Pickworth.

3. Floors

Until relatively recent times, conventional reinforced concrete slabs have constituted almost the only floors system used in skyscraper type buildings. It is easily realized that, apart from their undesirably large weight, these floors do not answer the functional needs and requirements of tier buildings which have just been discussed. Their use has been rapidly decreasing, although they continue to be employed for special reasons and sometimes in modified form. Thus, in one of the newest and most remarkable buildings, the 38-story

Seagram building on New York's Park Avenue, floors consist of 4 in. of reinforced, lightweight foam concrete, topped by a 3 in. lightweight concrete fill. All electrical ducts (for power, phones, intercommunication systems and internal television) are located in this fill. This is one example of the possible adaptation of reinforced concrete floor systems to the demands of modern skyscraper design.

The more modern floors systems, which have developed within the last 10 to 15 years, are very completely discussed in Mr. Stetina's contribution. In addition, Mr. Krapfenbauer devotes a large part of his paper to cellular floors, probably the most important of these newer systems. While, as Mr. Stetina points out, strength as a support was almost the only function in previous times, modern floors have in addition to satisfy a great variety of other requirements. In office buildings they must provide for complete flexibility of electrical and other services and connections, and for equal flexibility of locating and relocating movable partitions. They must furnish room for conduits and ducts for other services, such as heating and airconditioning, the latter having become an absolute necessity, particularly in largely glass-sheathed buildings. Reduction of dead weight contributes to structural economy, but brings with it sound transmission problems. Thus, acoustical conditioning, in regard to reflection as well as transmission, has become an important floor system requirement. In connection with reduced story height, provision for incorporating recessed, diffused lighting has assumed increasing importance. From the viewpoint of speed and economy of erection, elimination of the tedious scaffolding necessary for conventional reinforced concrete floors is a primary requirement. It is also desirable that floor construction be practically simultaneous with steel erection, story by story, and that the floor, when erected, be immediately available at full strength as a working platform for the other trades, such as plumbers, electricians, etc.

Mr. Stetina describes the various newer floor systems which satisfy some or all of these requirements. Thus, a newer development of reinforced concrete floors consists in slabs poured unto thin, profiled sheet steel forms. These forms are mostly self-supporting and thus eliminate temporary wooden forms as well as shoring. They stay in place in the completed structure and constitute the positive bending reinforcement of the slab. The floors are often made of lightweight concrete, and some systems permit the location of at least some of the necessary conduits within the slab.

Probably the most economical modern system from the viewpoint of first cost are steel bar joists supporting a thin concrete slab. The joists are simply standardized, mass-produced closely spaced, light trusses. The chords are made of light steel shapes, either hot-rolled or cold-formed from sheet steel, while the diagonals consist of round bars.

These two systems are economical and frequently used where ducting and conduit requirements are moderate, such as in hospitals, hotels, apartment buildings, and the like. They are rarely employed in large, multi-story office buildings. For these, cellular floors made by cold-forming of thin sheet steel represent the most important post-war development. Their wide acceptance would not have been possible without the results of another intensive and sustained research effort of many years' duration. These are the investigations which the writer and his associates have carried out at Cornell University on the strength and performance of cold-formed, thin-walled steel structures and which have led to the A.I.S.I. Manual and Specification for the Design of Lightgage, Cold-formed Steel Structural Members, the official American document in this field. The writer has reported on some phases of this work at the Liège and Lisbon Congresses and has given a comprehensive review of the entire field of thin-walled steel construction at the Cambridge Congress.

Several of the mass-produced, cellular floor panels are described and discussed in the contributions by Mr. Stetina and Mr. Krapfenbauer. The structural design of these floor elements, in regard to carrying capacity and deflection under load, is carried out by the engineering staff of the fabricator who mass-produces the particular type. The designer of a tier building merely selects the appropriate type from a catalogue. For this reason it is easily forgotten that the structural design of such thin-walled steel shapes by itself represents challenging and interesting problems of considerable complexity.

The features which have resulted in the wide acceptance of cellular floors are documented in detail in the contributions of Mr. Stetina and Mr. Krapfenbauer. Among them are the availability of the cells for a great variety of ducts and conduits, the installation, at full strength, of the flooring simultaneously with the steel frame erection, the absence of any formwork and scaffolding, the immediate availability as a working platform, etc. The writer would like to add that in cases where ducting requirements are minor, the cellular deck itself often constitutes the exposed ceiling. It is then acoustically treated by perforation and insertion of sound-absorbing light-weight materials, and provision is made for using part of the cells for recessed lighting.

From the viewpoint of structural economy it is important that, as Mr. Stetina remarks, cellular floor systems weigh only one-third to one-half of conventional concrete slab plus steel beam floors. This sizable reduction in dead load, naturally, permits corresponding weight savings in the main skeleton and in the foundations. It is one of the several influences which, as has been mentioned before, have greatly reduced the weight of the main framing of tall tier buildings.

4. Walls

Tall office buildings were first erected with bearing-wall construction, rather than with steel or concrete frames. Mr. Stetina points out that in the 1880's the bearing walls of 16-story buildings were up to 15 ft. thick at the base. With the development of skeleton construction, walls lost their bearing function and served exclusively for enclosure and insulation. Even then, masonry remained practically the only material until quite recently. One of the first buildings with an all metal-and-glass facade was the administration building of the Aluminum Company of America in Davenport, Iowa, 1948, and one of the first all-glass facades that of the United Nations Building in New York, 1949—1950. But the trend to thin walls in tall buildings received its strongest impetus from the erection, 1951—1952, of five big buildings in Pittsburgh, four of which have stainless steel facades and one an aluminum facade.

Present curtain wall construction is described in detail in Mr. Stetina's contribution, supplemented in regard to metal curtain walls by Mr. Krapfenbauer. Masonry continues to be used for curtain walls, but its weight has been reduced from about 130 to about 80 lb./sq. ft. Recently, masonry sandwiches have been developed which weighed as little as 30 lb./sq. ft. and consist of a thin, exterior natural-stone slab backed up by light-weight insulating material.

The main current trend, however, is to combinations of metal and glass, in various proportions, including such all-glass facades as the United Nations and the Seagram Buildings in New York. They are as thin as 2 in. and even less, as compared to the 12 in. of the standard masonry curtain wall, and weigh as little as 4—5 lb./sq. ft. as compared with 80 to 130. The consequence is another large item in reduction of total load on the steel skeleton, and a corresponding saving in steel and in foundation cost. Likewise, the increase in usable floor space adds to the rent income, a representative figure being 5% for a 100 ft. square building. Typical metal curtain walls are erected from the inside, eliminating the cost and complexity of exterior scaffolding. In addition, the great variety of textures and colors now available for facades has added a new dimension to architectural expression.

Not all is saving, however. In particular, the cost of some of the thin curtain walls is considerable. If the average cost per square foot of a brick curtain wall is designated as 100%, porcelain enamel steel walls cost approximately 150%, aluminum walls 170%, stainless steel walls 160 to 240%, and all-glass walls 220 to 250%. In addition, if a large proportion of the facade area is glass, difficulties with and costs of interior climate conditioning are considerable. In particular, large glass areas call for greatly increased air conditioning installations, with added initial cost and current expense. With large glass areas it may become necessary to heat and cool a building simultaneously. Even in winter time temperatures up to 140° on the south and west side have been measured simultaneously with temperatures as low as 0° F. on the north and east side. In a largely glass sheathed building in Los Angeles during the winter, inside temperatures will rise to 90° F. when the outside temperature is 50°, even though the air conditioning system is running at top power. This situation is better in the summer when the sunlight, impinging at a steeper angle from the high-standing sun, is mostly reflected from the glass surface. Similar difficulties have been encountered in all localities where large glass proportions have been used. It seems to the writer that the present fashion of excessive use of glass in facades, while capable of some striking architectural effects, is an aberration and will not last very long. It is not conducive, and even detrimental, to the well-being of the building occupants and its high initial and maintenance costs (in heating, air conditioning and cleaning) are not balanced by corresponding advantages.

Other initial difficulties with thin curtain walls have been: lack of weather tightness, condensation on the interior face of the external metal sheet, tight dimensional tolerances which made it difficult to adapt to the normal dimensional deviations of the steel frame, the problem of external cleaning of the immovable window and glass surfaces (solved by the use of movable cleaning platforms hung from the top of the building), and the like. All these have been successfully overcome and thin curtain walls are now prevalent not only for multi-story buildings but also for lower one- to three-story commercial, administrative and residential structures.

5. Fireproofing

Extensive changes have occurred not only in means and devices for fireproofing, but in the entire conception of fire protection. It is realized that buildings must be classified in regard to degree of fire danger and extent of necessary fire protection, depending on the "fire load" as discussed in Mr. Kollbrunner's contribution, on use of building, size of incombustibly enclosed area, and many other factors. At the same time, the fire resistance of various types of construction is now being determined by extensive and large-scale fire testing. As Mr. Boué shows, even without international standardization the various industrial countries have arrived at very similar fire-testing procedures, in regard to the imposed time-temperature curve and other features.

As an example of the development, Mr. Stetina recalls that the old 4-hour fire rating requirement for walls was originally meant to apply to bearing walls which, if weakened by fire, would cause the building to collapse. Carried over into framed construction, this provision made no sense, particularly in connection with the fact that no limits were set on the amount of windows which have essentially zero fire resistance. These obsolete requirements have now been changed and many American cities require only a 2-hour fire rating for external walls, while a number of others merely stipulate that the wall must be of incombustible construction. Analogous developments have occurred in regard to requirements on interior fireproofing.

Similar radical changes have taken place in means and techniques of fire protection. It used to be that steel beams and columns had to be fully encased in concrete. Such full encasement is now largely replaced by membrane fireproofing by means of light-weight plaster. This produces a weight saving in the ratio of about 15 : 1, as noted by Mr. Stetina. Cellular steel floors are being fire-proofed either by light, suspended plaster ceilings or by the spraying of lightweight plasters (various mixtures of vermiculite, perlite, gypsum, asbestos), directly onto the deck.

The weight savings which can be achieved in this manner are very impressive and add to those obtained through the use of modern floor and curtain wall construction. Mr. Boué notes a case where changes in fireproofing alone have resulted in a 20% reduction of construction costs.

6. Frame Design

While the structural engineer should, and does play a role in regard to the various features discussed so far, his main concern is, of course, the design of the main steel framing. In this regard, too, considerable changes have occurred, in part in view of developments in floor, wall, and fireproofing techniques, in part as a consequence of advances in the art of steel construction.

To begin with, it should be noted that reinforced concrete continues to play a role in the framing of tier buildings. For structures up to 10 or 15 stories, and even 20, it is not unusual that reinforced concrete is found more economical than steel. For taller buildings the columns in the lower stories assume excessive dimensions, but this can be alleviated by using composite columns in these lower floors, consisting of heavy, rolled steel shapes encased in concrete, This was done, for example, in 1957 for a 24-story office building in Cleveland which had originally been designed for steel framing, but was changed to reinforced concrete when a steel shortage threatened speedy completion. (The interesting measurements on a steel-framed building reported in the contribution by Messrs. Sparkes, Chapman and Cassell seem to indicate that under the low loads at which these measurements were made the concrete in the column encasement did not carry any of the loads. The brevity of this paper makes it difficult to interpret these findings since no information is given as to transverse spiral or other reinforcement of these columns and because the measured stresses were quite small compared to the full design stress, let alone stresses near failure. This merely illustrates the usual difficulty in interpreting strain measurements made on actual structures during construction.) In two successive years, 1958 and 1959, the New York City Housing Authority took alternate bids in steel and in concrete framing for groups of 20-story, low-cost housing developments. In both cases concrete framing produced a saving of some 6% in total construction cost.

This illustrates that reinforced concrete maintains its place mostly for functionally relatively simple structures (apartment buildings, hospitals, etc.) of moderate height and where low first cost is of decisive importance. However, the preponderance remains with steel framing, particularly for office and similar buildings. Thus, of 26 tall buildings (7 to 64 stories) under construction in New York in 1959, 24 were steel framed.

Until after the Second World War, riveting was almost the only connection method employed for steel framing, except for a limited use of bolts for secondary field connections. In the years around 1950 a definite trend toward the use of welded connections began to develop, particularly in localities at sizable distances from well-established fabricating shops. For instance, of 18 buildings of an average height of 18 stories, begun in Texas in the years 1951 to 1953, 12 were of welded construction and only 6 were riveted. The contribution by Messrs. D. T. Wright and R. M. Gooderham describes an interesting ten-year development of welded construction in Toronto, Canada, which began in the first buildings with the substitution of welding for riveting in simple, customary connections, and culminated in the latest structures in fully rigid, welded construction, with butt-welded connections used throughout. This may be an illustration of the fact that developments in one country are not necessarily conclusive for another. The writer doubts that under American conditions these butt welding methods would prove economical. While they definitely save material, the authors clearly describe the close dimensional tolerances that must be maintained and the special precautions which are taken to accommodate even the small deviations from ideal shape which are found in all structural members. These are complications which are probably tolerable where ease, speed, and cost of erection are not of decisive importance.

As Mr. Pickworth indicates, in recent years high-strength, high-tension bolts have become by far the most common means for main field connections, while ordinary unfinished bolts continue to be used for connecting secondary members such as floor beams, purlins, and the like. High-strength bolts have hitherto been designed on the rather primitive basis of simple one-for-one substitution for rivets of the same size. Impending changes in design codes will reflect more closely the results of yet another extensive research undertaking which has been done on these connections. These changes will permit the designer to utilize the inherent strength advantage which high-tension bolted connections have in several respects over identical riveted connections.

To be sure, there are exceptions to this trend. Thus, in the 19-story Inland Steel Building in Chicago, one of the most interesting recent structures, all main column and girder connections are welded and only secondary connections are bolted. The building is free of interior columns, the main girders spanning about 60 ft. between exterior columns which are located outside of the enclosing curtain walls. However, in a very similar structure erected two years later (1958), the 20-story Crown Zellerbach Building in San Francisco, whose girders span 63 ft. between exterior columns, all connections were bolted except for the welded butt splices of the main columns. The advantage of one method over another often depends on local and erection conditions. Thus, in 1959 in a 9-story building in Denver, the mere substitution of welding for high-strength bolting, without any changes in the sizes of the members, resulted in considerable savings both in cost and in construction time. Also, the 1958 Union Carbide Building in Canada, described by Messrs. Wright and Gooderham, is of very similar layout as the Crown Zellerbach Building in San Francisco but, in contrast to the latter, is all-welded. However, to date these are the exceptions; it is high-tension bolting which accounts for the majority of structures.

The radical reduction of dead loads which comes from the utilization of modern floor-, wall-, and fireproofing systems has been described. Taken in combination with improvements in design methods this has resulted in a very sizable reduction of the weight of the main steel framing itself. Statistics are difficult to obtain, but a tabulation in Mr. Stetina's contribution indicates that within the last 25 years the weight of steel framing, for buildings of identical size and layout, may have decreased to roughly one-half. These lighter frames are, of course, more flexible than the previous, heavier structures. In addition, the conventional masonry panel walls, reinforced concrete floors, and full concrete encasement of columns and girders in the older structures had all contributed greatly to the strength and, particularly, stiffness of the structure, even though this influence was generally neglected in design.

In contrast, today's light-weight floors and walls make no contribution to strength and stiffness (except when especially designed for the purpose). Consequently, questions of deflection, resistance to horizontal loading (wind, earthquake, etc.) and horizontal sway have assumed much greater importance than in previous years. It becomes more frequently necessary, therefore, to calculate vertical and horizontal deflections and even to carry out dynamic analyses in order to limit oscillations from vertical as well as from horizontal loading. These calculations, where possible, should be made for actual, expected loads, rather than for the legal loading usually prescribed in codes.

In regard to horizontal deflection Mr. Pickworth indicates that sway under design wind load should not exceed 0.15 to 0.20% of the height. For the previously mentioned Crown Zellerbach Building in San Francisco, sway under design wind load was limited to about 0.12% and for seismic load to about 0.16%. (Design for earthquake loading is mandatory on the Pacific coast and in some other parts of the USA. In the case of the Crown Zellerbach Building the equivalent seismic load was 3.5% of the vertical gravity loading.)

The manner in which horizontal loads are resisted has become of greater interest and concern. There are, essentially three ways for framed buildings to resist horizontal loads. The skeleton itself, designed as a rigid or semi-rigid frame, is dimensioned to withstand the horizontal loads. Or else, vertical truss bracing is introduced in selected planes to receive the horizontal loads. Finally, solid vertical concrete walls, known as shear walls, are also used in appropriately selected planes. They are usually arranged around the service core which contains the banks of elevators (lifts) and other services, but narrow exterior walls are also used on occasion. Rigid frame bracing gives the least interference with interior space requirements and, therefore, is preferred, but may result either in excessive deflections or in excessively heavy members. It is frequent, therefore, that special bracing in selected vertical planes, by shear walls or truss bracing, is advantageous or necessary. In this case the floor construction must possess sufficient strength and rigidity to resist horizontal loads and conduct them, in "diaphragm action", to the braced vertical planes. Extensive, full-size tests have shown that not only concrete slabs but also lightweight cellular steel floors are capable of developing the necessary diaphragm action, if the individual panels are adequately connected by welding.

The wind bracing design is among the more challenging problems of analysis. For unsymmetrical buildings precise calculations must be made to determine how much of the total horizontal force is resisted by each of the braced vertical planes. The problem becomes even more involved when several systems of different rigidity are used in the same structure. Thus, in the Crown Zellerbach Building vertical truss bracing is used in the separate service tower, which contains the elevators and other services, while the interconnected main building relies on rigid frame action to resist wind and earthquake forces. The 38-story Seagram Building has vertical truss bracing which, up to the 17th floor, is imbedded in 12 inches of concrete.

7. Erection and Safety

The erection of high, large tier buildings is just as impressive an engineering accomplishment as is their planning and design. It is not the mere fact that highly specialized and ingenious techniques are used, as described in Mr. Rapp's contribution, some of which amount in essence to lifting oneself up "on one's own bootstraps". In addition to these technical accomplishments, the construction of a vast building in the crowded center of a modern city makes the highest demands in regard to overall organization. There is usually no space available for storage or staging. A continuous flow of materials must be maintained, in closest co-ordination with the steel fabricating shop and other suppliers. Traffic to, from and on the site must be maintained in the face of all the outside interference which the big-city location implies. The problems of logistics are by no means inferior to those encountered by the military in directing large masses of troups.

In all this, the problem of workmen's safety is not a separate feature, but an integral part of the process. It is evident from Mr. Rapp's and Mr. Wolf's contributions that each phase, technique, and procedure must be planned both for safety and for technical efficiency. The American experience seems to show that any unsafe measure is also an inefficient and uneconomical measure. This experience also seems to indicate, as is evident from Mr. Wolf's paper, that a very high safety record is best achieved by a long and sustained safety tradition, by voluntary and intensive co-operation between employer and employee, management, foremen, and working force, rather than by the detailed imposition of safety laws by government agencies, enforced by official, outside inspectors. Again, the experiences in one country are not necessarily transferable to the conditions of another.

Rapport général

1. Tendances actuelles dans l'étude et le calcul des immeubles à ossature métallique

La majeure partie du présent rapport concerne les immeubles à étages multiples en charpente métallique du type bâtiments d'affaires, hôtels, immeubles locatifs et autres. Bien que d'importantes améliorations aient été apportées aux méthodes de calcul des ossatures métalliques, aucun changement fondamental n'est encore survenu dans ce domaine. Néanmoins un rapport concernant les ossatures métalliques se doit, à l'heure actuelle, de rappeler qu'une nouvelle méthode de calcul, foncièrement différente de l'ancienne, commence à se répandre tout au moins dans l'étude des ossatures formées de cadres soudés à un ou deux étages, comme celles des bâtiments industriels, entrepôts, etc.

Je veux parler de l'étude et du calcul basé sur le comportement plastique de l'acier de construction. Différents aspects de cette méthode ont été discutés en 1956 à Lisbonne et spécialement en 1952 à Cambridge. Depuis lors, en Angleterre et aux Etats-Unis, les règlements concernant l'étude et le calcul de constructions en acier autorisent officiellement l'application de cette méthode. C'est le résultat de plus de douze ans de recherches intenses et patientes effectuées de façon plus ou moins indépendante en Grande-Bretagne (Université de Cambridge) et aux Etats-Unis (Universités de Lehigh et Brown). Aux Etats-Unis, ces résultats ont été inclus à un appendice aux normes, officiellement reconnues, de l'Institut américain de la construction métallique. Cet appendice, adopté en décembre 1958, permet l'usage de la méthode de l'équilibre plastique et contient des prescriptions précises concernant son application. Celle-ci est limitée aux poutres continues ainsi qu'aux cadres rigides métalliques à un ou deux étages. En Angleterre et aux Etats-Unis, quelques ouvrages ont déjà été calculés d'après la méthode de l'équilibre plastique avant l'entrée en vigueur de ces prescriptions. Le nombre des immeubles calculés de la sorte ne cesse d'augmenter, bien que les deux méthodes continuent à exister côte à côte. La méthode plastique, appliquée judicieusement, se distingue par une économie d'acier, une diminution du temps utilisé pour l'étude et le calcul et par une plus juste conception du comportement effectif des constructions métalliques rigides sous l'effet d'une charge.

Les recherches doivent être poursuivies tout spécialement en ce qui con-