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

Band: 8 (1968)

Artikel: Plastic design

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DOI: https://doi.org/10.5169/seals-8705

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Tall Multi-Storey Buildings

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Plastic Design

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1. Introduction

1.1. Plastic design has taken an important part in Structural Engineering during recent years, but we must note that there doesn't exist a generally valid theory from the point of view of the Physicist. The basic equation for the continuum independent of the material are formulated, however we haven't found hitherto the general law for plastic deformation covering the relationship between stresses and strains (strength hardening included), temperature and its variation with time. The research done by A. R. Green and P. M. Naghdi [1] may be considered as a great step forward in formulating such a law. They furthermore made an attempt to develop a general theory of plasticity starting from the basic laws of continuum mechanics and maintaining only a few restrictions.

1.2. Coming back to the theory of structures we note, that the plastic working capacity of the material used in structures has long been utilized to establish the load carrying capacity. Above all in the field of stability-investigations the plastic reserve, dependent upon the shape of section, has been considered theoretically as well as experimentally mentioning here only the researches of L. von Tetmajer [2], F. Engesser [3], M. Roš and J. Brunner [4]. Also for beams stressed preponderantly in bending—especially continuous girders—the plastic design has been applied to simplify the calculation by supposing an elastic-ideal plastic material. We mention here only the works of F. Stüssi and C. F. Kollbrunner [5], G. Kazinsky [6], N. C. Kist [7] and H. Maier-Leibnitz [8].

To the application on high strength steels H. BEER and K. MOSER [9] have based the limit design on an arbitrary strain-stress diagram, while CH. MASSONNET [10] considers the application of the theory of plastic design on type 52 steel.

2. Suppositions and Limits of Plastic Design

2.1. In the practical dimensioning of some types of steel structures we can partially or completely leave the basis of the theory of elasticity for economical reasons and make use of the theory of plasticity. The plastic design gives generally a better criterion of the load carrying reserve of the structure than may be obtained by the theory of elasticity (dimensioning with permissible stresses). To handle with sufficient safety the method of "plastic design" some suppositions must be fulfilled of which the most important may be mentioned as follows:

2.2. The material must have a sufficient plastic working-capacity; it is often possible to simplify the $\sigma - \varepsilon$ diagram in an ideal-elastic and ideal-plastic part (easy to represent analytically). Due to the restriction of the deformations ($\varepsilon_{def} \approx 15 \varepsilon_F$), strength hardening is not considered. The supposition of an ideal elastic-plastic strain-stress diagram is sufficiently valid for the annealed structural steel. Having a bar with residual stresses, the coupon test shows clearly a premature curvature of the strain-stress-diagram, the limiting plastic moment depends very much on the magnitude and the distribution of the residual stresses over the section.

2.3. We must dimension the selected sections so as to avoid local instability (buckling of the sheet, lateral buckling). In steel-skeletons an efficient restraint against lateral buckling is often available by a suitable design of the floor and wall structures.

2.4. Calculating the limit load we must separate the deformations into elastic and plastic ones. Considering a multi story frame with lateral sway we have—strictly speaking—to verify the stability against buckling, and to calculate the deformations according to the elastic theory of second order after the formation of every new plastic hinge; then to calculate the additional plastic deformation and confront it with the corresponding permissible deformation, before admitting further plastic hinges. This is because many calculation methods have the restriction that instability phenomena have to be excluded till the formation of a plastic hinge mechanism, and this may not be fulfilled sometimes in practice. The problem of the "divergence of equilibrium" caused by a change of the buckling mode as well as the consideration of the possible buckling perpendicular to the plane of the frame has not been investigated due to the excess of calculation work, by which an important advantage of plastic design will be lost.

2.5. Considering the formation of plastic hinges, especially of multi-story frames, we must take into account that the joints of the stanchions and the beams are not rigid and that it would therefore be important to include in our calculations the elastic restraint of these joints. This is valid for welded frame joints as well as HSFG-bolts front plate connections. J. OXFORT [11] and U. VOGEL [12] proved, that the spread of plastic zones near the plastic hinges is only of some influence—compared with the usual supposition of local hinges—in sections with a great plastic reserve, which are rarely used in steel structures. However it may become of increasing importance for a flat diagram of bending moments having a great longitudinal section in the partially plastic range. H. BEER and K. MOSER [9] have shown how one can include these zones by distorting the moment-diagram.

2.6. Considering the calculation of frame-work it is important to underline that, applying plastic design, we cannot superpose the loading cases with different sequence of the formation of plastic hinges.

3. Calculation Methods for Plastic Design (2nd order) of Multi-Story Frames

3.1. At the 8th Congress we have to deal only with the skeleton of high, slender sky-scrapers, preferably without interior columns, with or without floor- and wall-slabs. Before entering in these special types it may be convenient to come back briefly to the development of plastic design of multi-story frames and to pass then to its application to the types of structure considered here.

Applying plastic design we have to distinguish basically between, the "simple" problem (i.e. beams primarily stressed in bending) and the load carrying problem considering the stability (i.e. multi-story frames). In the latter case the investigations have to be carried out according to the theory of second order in the elastic as well as in the plastic range.

While the calculation by the elastic theory of second order has been essentially simplified using the tables of the stability-function of E. CHWALLA [13] and E. SCHABER [14], the investigation in the plastic range requires a great amount of calculation work. It is therefore convenient to try to obtain a first estimation of the limit load using the theory of elasticity of second order and the limit design theory of first order. According to N. DIMITROV [15] we have:

$$P_{el} < P_{cr} < P_{pl} < P_k$$

where:

P_{el} = the maximum load according to the theory of second order characterized by the first yielding in the borders of the section (calculated conveniently by the above mentioned tables of the stability functions);

P_{cr} = critical load, limiting the load carrying capacity (Collapse load);

P_{pl} = ultimate load according to "plastic design" supposing a mechanism (Theory of plasticity of first order);

P_k = ideal-elastic buckling load due to an antimetric buckling figure.

3.2. The approximation of the critical load by elastic and plastic calculation methods has been realized by M. R. HORNE and W. MERCHANT [16] presenting an empirical Rankine-formula to calculate the critical buckling load of a framework in the elastic-plastic range:

$$P_{perm}\left(\frac{1}{P_{cr}}\right) = P_{perm}\left(\frac{1}{P_k} + \frac{1}{P_{pl}}\right)$$

respectively $\dfrac{1}{\nu_{cr}} = \dfrac{1}{\nu_k} + \dfrac{1}{\nu_{pl}}$ or $\nu_{cr} = \dfrac{\nu_{pl}}{1 + \frac{\nu_{pl}}{\nu_k}}$

P_{perm} = permissible load;
ν_{cr} = critical load factor against buckling in the elastic-plastic range;
ν_k = critical load factor against buckling in the elastic range (an approximate calculation is sufficient, because $\nu_k > \nu_{pl}$);
ν_{pl} = load factor which corresponds to the compressive yield point load (supposing constant section of the structure).

W. MERCHANT [16] proposes—as an improvement—to introduce a modified ideal plastic load instead of the compressive yield point load, which considers the deformations of the adjacent bars, still in the elastic range.

Many tests have confirmed this empirical formula, however its application is limited to such structures for which the buckling figure corresponding to the first critical loading step in the elastic range coincides in its shape with that of the ideal plastic mechanism.

The above mentioned authors justify the application of this empirical formula with many possible "imperfections" whose influence can only be determined on a statistical basis by an extended test program. Further test results in this field would be of great value.

R. H. WOOD [17] agrees with the application of this formula but he proposes to include also the local loading arrangement and to limit its application only to frames bent around their rigid axis. Furthermore, he feels that if there is a small critical elastic load there will be an overestimation of the reduction of the real ultimate load. A. HRENNIKOFF [18] criticizes, among other things, the incompleteness of the formula due to the neglect of the lateral torsional buckling as well as local instability (which M. R. HORNE and W. MERCHANT [16] obviously did not intend to cover) and doubts in some aspects the reasoning of the RANKINE-formula.

3.3. A further simplified calculation method has been presented by W. MERCHANT [19] and, in an extended form, by J. OXFORT [20], considering the load-deflection diagram plotted in Fig. 1. Using Young's modulus until the yielding point $\sigma_F = P_F/A$ we achieve the maximum possible loading capacity with this yielding stress. Since we require at that limit a safety-factor ν_F we still have until collaps, a safety-reserve $(\nu_{cr} - \nu_F)$ depending on the following influence factors: the plastic reserves in initially lesser stressed parts of the whole structure, the relation between the bending-, normal- and shear-stresses, the real strain-stress diagram of the constructional steel and the residual stresses in the sections. If we calculate a structure according to the following: the theory of elasticity of first order (curve 1), the theory of elasticity of second order (curve 2) the theory of plasticity of first order considering plastic hinges (curve 3) and finally the theory of plasticity of second order (curve 4) we obtain qualitatively the curves plotted in the load-deflection diagram. The intersection point G of the curves 2 and 3 may give a first approximation for the critical ultimate load. W. MERCHANT [19] proposes to provide the deflection (according to curve 2) with an amplification-factor (i. e. $n = 2$) such as to obtain a flatter course of the load-deflection diagram, but the factor $n = 2$ can only have limited validity. W. MERCHANT [19] shows furthermore by a serie of tests, that the relation P_{cr} to P_G is nearly constant, coming to the following formula:

$$P_{cr} = \beta P_G .$$

This may be often satisfactory but in some cases P_G is near P_F, so that P_{cr} would be smaller than P_F and that is contradictory. Therefore OXFORT [20] proposes to write the formula as follows:

$$P_{cr} = P_F + \alpha(P_G - P_F) .$$

The value $\alpha = 0.5$ proposed by the author has still to be checked with further ultimate load calculations and tests and it would be desirable to obtain corresponding contributions at the 8th Congress.

3.4. U. VOGEL [12] develops an iteration process with the restricting suppositions according to the points 2.2 till 2.4 of section 2. Starting from an arbitrary yielding mechanism of a frame structure he formulates the ultimate load condition according to the theory of plasticity of second order stating a non-linear system of transcendent equations for the calculation of the "critical loading factors" by exploiting the conditions of equilibrium and compatibility. This system of equations which cannot be solved explicitely is solved by an iteration process realizing the first iteration step with the known "trial proceeding" or the "combination of kinematic chains" (according to the plasticity theory of first order). A control of the deformation to check the place of the ultimate plastic hinge (whose supposition is contained in an equation of the system) as well as a statical control to check the correctness of the solution have to be done at the end of this investigation.

The results of this approximate calculation have been compared with the exact calculation of J. OXFORT [11] and are on the unsafe side up to 5% for **I**-sections (relative small plastic reserves) and up to 15% for rectangular sections (great plastic reserves), however this fact can be compensated by introducing a higher factor of safety (according to the proposal of the above mentioned author). The composition of the process makes it appropriate for computers.

In the Lecture Notes and the accompanying Design Aids [33], published in 1965, concerning "Plastic Design of Multistory Frames" further iteration processes are given, based on two different ways:

a) A preliminary design is done only for vertical loads and then the deflections caused by the vertical and wind loads are calculated. After checking the stresses of the first step plus the additional stresses of the second step the required sections of the struts and beams can be found with a further iteration process.
b) The guessed deflections for the bars are supposed and the design is done taking into account the acting loads and these deflections. Follows a further iteration step according to the ENGESSER-VIANELLO-method until the given deflection coincides nearly with the calculated values.

The preceding exposition shows that the investigation of the load carrying capacity of the skeleton can be considered in no way as finished and that therefore contributions to these problems, considering the four types of structure presented in section 4, will be very useful.

3.5. The situation is simpler in the case of structures with a stiffening core which, in compound action with the floors, provide a support of the multistory frames against lateral sway. This type of structure has been investigated by W. PELIKAN and U. VOGEL [21] among others. If we suppose that the horizontal forces (i.e. wind) are conducted completely in the core, the stanchions will be stressed, apart from local wind stresses, only by vertical forces and restraint moments due to the rigid joints with the concrete slabs. This restraining action can be produced by a surface-support without hinges on the foot and head plates or, by having a continuous stanchion concreted with the floor slab. Special horizontal frame beams can be arranged or omitted. In the latter case the stanchions in compound action with the floor slabs resist the loads.

It is known from the theory of plasticity that the stanchions can be calculated in both cases supposing hinges at their ends because they don't lose their load carrying capacity under the formation of plastic hinges. A calculation method for this special type of stanchion has been developed by U. VOGEL [12/21] introducing as additional stressing of the stanchion, besides the normal forces, an angle of end rotation, which can be calculated approximately as end rotation-angle of the freely supported floor slab, since the stiffness of the concrete floor slab is very great compared with that of the steel stanchions.

U. Vogel starts from the state of equilibrium of the system or the stanchions at the moment of reaching the critical load and states, on the deflected structure (according to the theory of second order), an implicit equation to calculate the critical stress relation $\varkappa = \sigma_{cr}/\sigma_F$ in function of the slenderness. He evaluates this results with diagrams for **I**-profiles and rectangular sections. Considering middle slendernesses ($20 < \lambda < 60$) and small angles of rotation (that means concrete floor slabs $l < 7$ m) he shows that we can obtain by the plastic calculation method a little more economical solutions compared with the calculation supposing hinged stanchions. These results are also confronted with more exact calculation methods and checked by test results, arriving at a good agreement.

American scientists [33] particularly followed this way to obtain a dimensioning of the bars, calculating the angles of rotation of the bars for the steel structure only, on subassemblages. In this connection plastic hinges can be forced in the horizontal beams by a corresponding dimensioning.

3.6. From further calculation methods for multi-story frames we mention here the so called "Cambridge method" of R.H. Wood [17] which considers the problem of buckling stability but supposes frames without lateral sway. On the one hand the horizontal beams must be dimensioned such as to form plastic hinges close to the stanchions while on the other hand the dimensioning of the stanchions had to be done so as to remain with the stress-verification in the elastic range, which will influence the economy.

It would be of interest to communicate at the Congress experiences gained with these calculation methods and to present test results as well as to propose, if possible, improvements of the calculation.

3.7. Finally we draw attention to the importance of developing calculation methods appropriated for a computer (particularly iteration processes). Many calculation methods well known in structural engineering are not fitted for computer calculation because the relation between "put in" time and computer calculation time impairs strongly the economy of the computer. Therefore we have to think, when developing new calculation processes, simultaneously of the use of computers.

M.R. Horne and K.J. Majid [22] have developed an iteration process to calculate multi-story frames (also with inclined bars), taking also into account phenomena of instability particularly appropriate for electronic computers. The above mentioned authors start from a matrix-equation which comprises the loads, the compatibility conditions, the sways of the bars and the displacement of the joints. It is shown that even supposing a linear relation between load and bending moment, good convergence is maintained in the first approximation. They demand as a restriction that no plastic hinge-rotations occur under service load and that also no plastic hinge is produced in a stanchion after multiplying the loads with the permissible loading factor (i.e. $\nu = 1.4$ loading case II corresponding to the English Standard). A more exact elastic-plastic calculation must be carried out after this iteration to check the load carrying capacity of the frame.

4. Discussion of the actual construction-methods for multi-story buildings, considering the possible application of plastic design

4.1. Actually the multi-story frame with lateral sway (Fig. 2) is calculated in practice according to the theory of elasticity of second order. The publications of W. MERCHANT and M. R. HORNE [16], E. CHWALLA [13] and E. SCHABER [14] intend to comprise the plastic range introducing instead of Youngs modulus of elasticity a reduced value such as the value $E_1(\sigma)$ suggested by F. R. SHANLEY [23] or the buckling modul $T(\sigma)$, according to ENGESSER [24] supposing in the same way the shear modulus G in the plastic range dependent upon the Modul $T(\sigma)$ or $E_1(\sigma)$ if they consider the shear-deformation.

Now newer calculation methods partially try to realize stability investigations on systems transformed by the formation of plastic hinges with approximated and iteration processes (comp. 3.2–3.4, 3.7) using strongly limiting suppositions. Here, the investigation of systems transformed by plastified sections is of great importance, especially before achieving the ultimate load (mechanism). H. BEER [25] presents a corresponding investigation of a frame-joint of several bars taking into account all imperfections and an arbitrary strain-stress diagram.

The problem of the divergency of equilibrium according to the theory of second order and supposing a variable deflection figure (as mentioned in section 2.4) has not been studied till now. Here we have to check if the buckling load is achieved before the mechanism of plastic hinges is formed. For this proof already E. CHWALLA [26] proposes a substitute load, supposing that all loads are applied in the joints, and carrying out a stability investigation with this type of loading. Of course we can hardly speak here of a simplification, since we must add to the calculation according to the theory of plasticity also a calculation with the theory of elasticity.

4.2. Another construction-method for tall multi-story buildings is characterized by supporting the frames elastically with horizontal floor slabs (concrete or bracings) in all or only some floors, transmitting these horizontal forces to vertical slabs or bracings arranged only on the gable-sides or also between them. While we represent in Fig. 3a a skeleton with a central core, Fig. 3b shows the support by two cores, intended for large buildings. In Fig. 4a two vertical latticed slabs are arranged to provide an elastic support for the frames. We obtain the scheme represented in Fig. 4b and have to calculate the spring-factor as a non-linear function of the stiffness-relations and the loading of the horizontal and vertical slab-system.

In this connection the problem of the horizontal load distribution between slab- and frame-system first arises, before entering in the ultimate load calculation itself and the developement of calculation methods for the frame skeleton. Comparative calculations of the stiffnesses of a single bay-two story frame with diagonal bracings according to the theory of second order have shown that, supposing a horizontal force of 10% of the vertical forces (acting in the nodes), the latticed slab is 8 times stiffer than the framework. In the plastic range the frame may be even more flexible but having vertical slabs smaller than the whole width of the building (which may be arranged on staircases or lift-pits) the stiffness-relation can change fundamentally. In the latter case the relation of horizontal frame stiffness to horizontal slab stiffness can be as high as 1:3.

The application of ultimate load design to frame-work has already been discussed. However the application of the ultimate load design to latticed structures will be cleared up in connection with the box type (see 4.3).

The reduction of the lateral sway of frames can also be caused by the wall- and floor slabs. R. H. Wood [17] has proved this fact by tests and calculations of the compound action as well as reports on buildings. The influence factors which increase the stiffness can be divided into three groups:

a) The variation of beam loading due to the redistribution caused by floor slabs and walls. If, for weak beams, the loading intensity in the field decreases, as has been proved by tests, different restraining moments are obtained from the loads of these beams, which will reduce the real stresses in the stanchion compared with those obtained by ordinary calculation (see also the corresponding French Standards).
b) The increased stiffness of the beams against rotation of the joints due to neglected compound action of the floor-plate or an encasement with concrete.
c) The reduction of the lateral sway due to the compound action of the frames and wall-elements. Tests have proved that there is a considerable reduction of lateral sway and an increasing carrying capacity of the skeleton if the compound action between walls (also having light weight walls) and adjacent frames is established.

R. H. Wood [17] believes that the compound action between skeleton and surrounding encasement would be able to cover the difference between the effective buckling load of an elastic-plastic frame and the buckling load calculated according to the theory of plasticity of first order. No doubt that the supporting forces of a frame with lateral sway to assure the lateral stability are only small. However, these observed effects and the existent test results need further confirmation. It would be very important to deal with such investigations at the 8th Congress, giving much value to observations and surveys on existent constructions.

4.3. If a construction consists only of slab-elements joined with shear connectors, and these slabs form a box in which at less three slabs are not parallel; we will call it "Box-type". Fig. 5a shows a building surrounded by four latticed walls passing the diagonals through several floors so that the floor slabs and intermediate stanchions transmit the loads to the joints of the latticed walls. In Fig. 5b the design with framed walls is represented. These multicellular frames can be substituted statically for slabs although their stiffness is significantly lower than that of a full slab.

Using computers it should be possible to calculate such frame-works taking into account also the stability and representing its action in the whole system under certain conditions by introducing fictitious slabs of equal bending-shear deformation in order to simplify the calculation.

It is obvious that cores with exterior wall slabs can also act together to resist horizontal loads establishing the spatial static action by the floor slabs. Here more research work must be done applying plastic design to the following problems: frame action of latticed systems, reduction of the effective length of bars in latticed systems by the restraint action, more exact consideration of the different deformations of frame-work and latticed system having a more or less stiff core (as has been mentioned shortly in section 4.2).

4.4. From the described systems, the type with resisting core represented in Fig. 3 is very frequently applied today. Beside many constructional reasons this is probably due to the clear statical system because the wind forces are taken nearly completely by these cores so that the surrounding frames can be considered without lateral sway.

There are three possible plastic mechanism for these frames without lateral sway:

a) beam mechanisms, weak beams and strong stanchions;
b) sway mechanims, strong beams and weak stanchions;
c) the stiffness of beams and stanchions is about of the same order.

One can calculate the cases a) and c) according to the ultimate load theory of first order because the influence of the deformation on the ultimate load of the beam mechanisms can be neglected due to the small normal forces in the beams. However the theory of second order must be applied if the stanchions collapse first (case b).

5. Repeated loading

5.1. Two cases have to be considered which can cause the collapse of the building:

a) fatigue of the material having a great number of loading cycles;
b) instability due to an increasing permanent deformation with every loading cycle.

5.2. For the case under consideration of tall multi-story buildings the failure of a building due to fatigue can only occur in very exceptional cases, i.e. if tools and equipment stress some structural members with a great number of loading cycles. Generally the wind pressure will not cause fatigue of the material, because we suppose so high static wind loads, that their number of loading cycles (complete loading and unloading and change of the wind direction) is relatively small. Theme IIIc will deal with the problem of whether fatigue of the material can be produced by vibrations caused by wind impulses. The variation of the service load in buildings used for offices, residences and shops will also generally not cause fatigue of the material because we do not have to reckon with many daily loading cycles (complete charging and discharging), however a special use of some rooms can stress to fatigue some parts of the building.

A particular investigation of this problem is not needed at the Congress.

5.3. The deflection instability is originated by the repeated plastification of some sections causing a progressive increase of the deflection of the skeleton until collapse takes place. Before achieving the load which will cause the deflection instability, the system will behave elastically, after an initial slight increase of the permanent plastic deformation. The system "shake-down" these permanent deflections. M. GRÜNING [27] has first mentioned this fact. E. MELAN [28] has applied his investigation to arbitrary statically indeterminate systems and B. G. NEAL [29] and M. R. HORNE [30] have furthermore extended it. The load which creates deflection-instability depends upon the static system and upon the value of the plastic moment in some determining sections. Generally "shake down" takes place also in the skeleton of tall multi-story buildings so that the deflection-stability of the structure is maintained. However special investigation must be carried out in cases of doubt to assure the deflection-stability of extraordinary types of frames. Hereby we have to take into account also the residual stresses due to the rolling and welding process.

This theme may also be treated at the Congress examining the fact if, and under which conditions, deflection instability in tall multi-story buildings can be possible.

6. Deflections due to static load

6.1. The question, of whether the utilization of the structure is finished due to inadmissible deflections, has to be considered from the point of view of flooring and walls as well as the special use of the building. Inadmissible deformations of the skeleton can cause great damage to the wall and glass front. The deflections caused by dynamic influences and its consequences will be treated in Theme IIIc.

6.2. Several processes had been developed to calculate the deflections in the elastic and plastic range. If the sequence of the formation of the plastic hinges is fixed the step-by-step deflection method may be of special advantage. Generally, inadmissible great deflections, which will limit the utilization of the building, will occur only on loading steps just before the formation of a mechanism, however further investigations on some types of structures may be of interest. Here we have to study also the question of whether we can reduce the margin of safety against inadmissible deflections compared with the margin of safety against collapse of the system due to the formation of a mechanism or deflection instability.

The strain stress diagram allows us also to estimate the loading step at which the first plastic hinge will probably occur. An exact calculation of this loading step is not possible since the load factor includes the uncertainties of the suppositions for the loading and the calculation as well as for the dispersion of the yielding stresses due to the inhomogeneity over the section and the residual stresses. However there is no objection against a plastification under service load of some fibres of a section for we can consider local plastifications in steel structures as a characteristic property and an efficient help against stress accumulation.

It is recommended that particular investigations be presented and that the problem be discussed at the Congress.

6.3. As has already been mentioned in section 2.2 the adoption of plastic hinges, which remain effective until collapse, supposes that the yield range is sufficiently extended to allow the forced hinge-rotation. In some cases a special investigation is needed to check this supposition.

7. Safety and loading factor

7.1. The loading factor has to cover the difference between the suppositions and idealizations for the calculation and the dimensioning and the really existing conditions. Therefore it has to consider all uncertainties of: loading-suppositions, calculation-bases, fabrication and assembling, and properties of the material. This problem has been treated at several Congresses, so that we mention here only some aspects of tall multi-story buildings.

7.2. Loading-suppositions: While the dead load of the structural elements and the flooring, walls and roofing can be calculated generally with sufficient exactitude, considerable uncertainties exist in the statement of the service load. Certainly, we dispose of statistical material of the real service loads considering the normal use of buildings but we can only fix values which are not exeeded with a certain probability. Therefore we must provide a reserve in the loading factor against possible overloading according to the corresponding probability. The European Convention of Steel Constructors recommends that the Standard service load be provided with a factor 1.33.

For high skeleton buildings the snow load is only of importance for the roof, however the wind load can become a decisive importance for the whole structure. The Standards of the different countries usually take the maximum wind gathered from meteorological observation, which however cannot be considered as catastrophical wind. Here we must provide also a factor which has been stated to 1.5. However this factor seems to be a little conservative considering the fact that the additional stiffening effect of cladding and walling is not taken into account generally in the static calculation. The stresses produced by a variation of temperature can accelerate the formation of plastic hinges and reduce the stability of the structures but these stresses do not affect the calculation of the ultimate load according to the theory of first order.

7.3. It may be convenient to treat together the incertitude of the basis of calculation and the inexactness of fabrication and assembling. Speaking of the basis of calculation we are refering by the way to sect. 2. Considering the construction we suppose generally a rigid restraint of the stanchions in the foundation which is really more or less elastic. The joints of the frames are considered rigid and this is also not the case in reality. O. STEINHARDT [31] has shown with the example of the HSFG-bolted front plate joint, that the elastic restraint of the joints can reduce the corner moments up to 15%, increasing simultaneously the corresponding field moments.

Certainly, these calculation-suppositions on the unsafe side are widely compensated by the stiffning effect of cladding and walls, so that we are on the safe side considering in the static calculation only the frame-structure. It must be studied in every given case if a factor of incertitude, which will not be much greater than the unity, has to be introduced.

7.4. It is known that the yield-stress in rolled **I**-beams is not constant over the section, having a considerable difference between web and flange. Particularly in compressed components this dispersion in the yield-stresses has an influence on the deflection and the stability, reducing generally the buckling load. This incertitude can be taken into account supposing a fictitious buckling modulus and eliminating then a special factor of incertitude; however we must consider that incertitude, calculating the plastic moment.

Since the thicker flanges generally have a smaller yield stress than the thinner webs we must introduce a factor depending upon the shape of the section. Residual stresses due to the rolling and welding process can also influence the formation of plastic hinges, as has been mentioned before. T. V. GALAMBOS and R. L. KETTER [32] have worked out graphs on the basis of supposed residual stresses.

8. Summary

Plastic design and the theory of ultimate load design is by no means complete. In this paper the state-of-the-art for the different types of tall multistory buildings i.e. the frame-skeleton, with and without resisting core, (or wind bracings) and the box-type structure, is exposed, underlining the problems to be treated at the Congress.