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# Progress in the Design of Steel Plate- and Box Girders

Progrès dans le dimensionnement des poutres à âme pleine et en caisson en acier

Fortschritte in der Dimensionierung von Vollwand- und Kastenträger in Stahl

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## 1. INTRODUCTION.

It is not the first time that the undersigned has the duty to report about progress in the design of plate and box girders in steel. He was invited to establish the General Report in this theme at the New-York 1968 Congress of IABSE A7 and, in 1971, at the request of the late professor BEER,at that time chairman of Commission 2 (Steel) of IABSE, he organized in London, jointly with professors L.S. BEEDLE and K.C. ROCKEY, a Colloquium entitled :

*Design of plate and box girders for ultimate strength*

whose proceedings, published in July 1972 |C7| , were favourably received by the members of IABSE. This important source of references will be mentioned in the references as : IABSE London 1971 Colloquium.

This field of research is still in a state of rapid evolution. One main reason is the big accidents which struck four large steel box girders during their erection(|B1| to |B9|). These accidents triggered an important research effort in the countries most concerned, namely Great Britain and the Federal Republic of Germany with some additional work in Australia, Belgium, Japan and other countries. In Great Britain, a Committee, called the Merrison Committee, was appointed by the government and published its conclusions | D4 | in 1972, in a form usually called the Merrison rules. The research on the strength of box girder bridges continues very actively in Great Britain, with the central activity at Imperial College, London. This Institution has organised from July 6 to 9, in consponsorship with the Institution of Structural Engineers, the Royal Institution of Naval Architects and Constrado an International Conference on Steel Plated Structures. The 22 papers cited from this source will be abbreviated in the references as "ICSPS paper N° X".

In Western Germany, a special subcommittee called "Unterausschuss "Stabilität" and especially its Working Group "Plate Buckling" was set up. The author participates, as foreign member, with professor P. DUBAS, to the work of this Committee, while a series of experimental and theoretical researches are still under way in Darmstadt, Hannover, Karlsruhe, Braunschweig, Bochum and Berlin. Finally, the author has the responsibility to lead a subcommittee of Committee 8 (Stability) of the European Convention for Constructional Steel-work (ECCS). This Work.Group (8/3) devoted to the study of buckling of plates has, up to now, concentrated his efforts on the most urgent problems, namely the design of plate and box girders. His work up to June 1976 was summarized into two State-of-Art Reports that have been included in the

*Manual on the Stability of Steel Structures*

which forms the Introductory Report of the Triple

*Colloquium on the Stability of Steel Structures,*

which will take place in Tokyo (9.9.1976), Liège (Belgium) (13 to 15.4.1977) and Washington (17 to 19 May 1977) under the Joint Sponsorship of IABSE, ECCS, USSRC and Japan Stability Research Committee.
I shall try, in 25 minutes, to present you a broad survey of these extremely complicated problems. My written report will be much more complete that my oral presentation. In a subject as fluid as this one, you could not hope that I shall come out with clear-cut design rules. I shall be happy if I can review thoroughly enough the recent research literature in the field concerned, stress the progress already obtained, and show the trend of the current research effort.

## 2. CRITICAL APPRAISAL OF THE LINEAR BUCKLING THEORY OF PLATES AND THE LESSONS TO BE DRAWN FROM THE ACCIDENTS TO BOX GIRDER BRIDGES.

The linear theory of buckling of plates is based on the following main assumptions :

1) the plate is initially perfectly flat ;
2) the transverse displacement w at buckling are so small ($w/t \simeq 0{,}2$) that membrane stresses in the median plate are negligible.

Both these assumptions are wrong but, nevertheless, the linear theory can provide useful results *provided the safety factor* is selected on the basis of the relevant experiments.

The main practical outcome of above theory for stiffened rectangular plate panels is represented by both books by professor KLÖPPEL and his collaborators |A1| |A2|, in which the charts are restricted to simply supported plates reinforced by open section stiffeners. The author and his staff have generalized KLÖPPEL's mathematics for orthotropic plates reinforced by closed section stiffeners (|A4|, |A9|) and verified that this type of stiffeners are vastly superior to open section ones |A5|.

However, there is to-day a very large consensus that the linear buckling theory of plates is not an adequate support for the design of plate-and box girders (|A3|, |A9|).

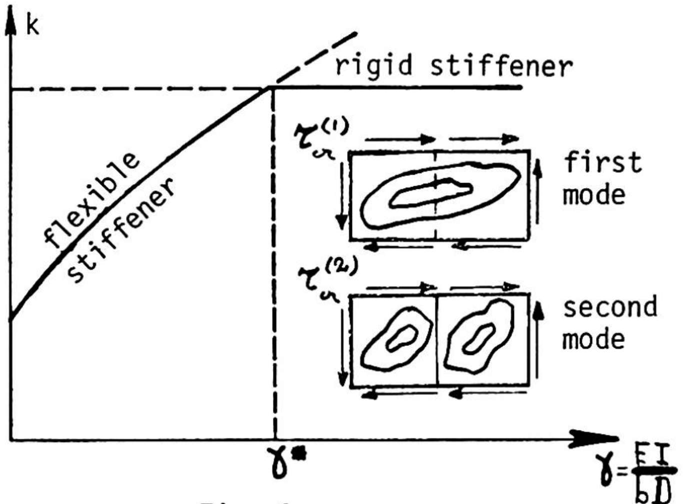
The New-York IABSE Congress of 1968, in his conclusions, already stated |A10| that "The linear theory of plate stability is not an adequate basis for design of struts and girders consisting of thin-walled sections"....
"New carefully planned test series and computer simulation,"
including the post-buckling behavior (membrane action) of the sheet",
are therefore strongly recommended".

This opinion is based on the basic philosophy of the design rules which has been prevailing these last years. The philosophy generally accepted in Europe, for concrete as well as for steel, is the semi-probabilistic theory of limit states,in which the adequacy of any structure must be controlled in two limit states:

a) the serviceability limit state (under working loads and other actions)
b) the ultimate strength limite state (under factored loads and other actions).

Now, if the linear buckling theory predicts accurately the bifurcation buckling load of a perfect plane panel, it does not tell anything about the postbuckling strength of this panel. This is dangerous because this postbuckling strength depends very much of the rigidity of the frame surrounding the plate and still more on the type of stressing (shear, bending or compression). Therefore, low safety factors derived from the shear case where it is known since the WAGNER studies |C2| that the strength reserve can be enormous, may be-and have been - misleading if they are applied to compressed panels.

There is another, even more dangerous defect of the linear buckling, which pertains to the concept of optimum relative rigidity (Mindeststeifigkeit) $\gamma^*$ of the non-dimensional factor $\gamma = EI/bD$ (where EI is the flexural rigidity of the stiffener and $D = Et^3/12\ (1 - \nu^2)$ the rigidity of the plate). The linear theory predicts that such a stiffener has fifty chances on hundred to remain flat under the critical load (say $\tau_{cr}$, fig. 1), and thus to enforce the second buckling mode (Mindesteifigkeit erster Art) or, at least, a mode where the stiffener remains *almost* flat (Mindesteifigkeit zweiter Art). As, theoretically, there is no benefit to increase $\gamma$ beyond $\gamma^*$ this notion has been extensively used (|A1|, |A2||A4|) to design stiffened plated structures. However, this concept has nothing to do with postbuckling strength. 22 years ago, the writer demonstrated experimentally |A3| that $\gamma^*$ stiffeners invariably bent in the postbuckling range if not sooner and recommended to multiply the theoretical value $\gamma^*$ by a coefficient m equal to 3 to 7 according to the position of the longitudinal stiffener, in order to obtain a stiffener quasi-straight up to collapse. He insisted that an increase of about 4 on $\gamma$ meant roughly an increase of 2 on the cross section area, and that the additional expense was very small, because the price of stiffener depends very much more on fabrication, adjustment and welding than on its weight.



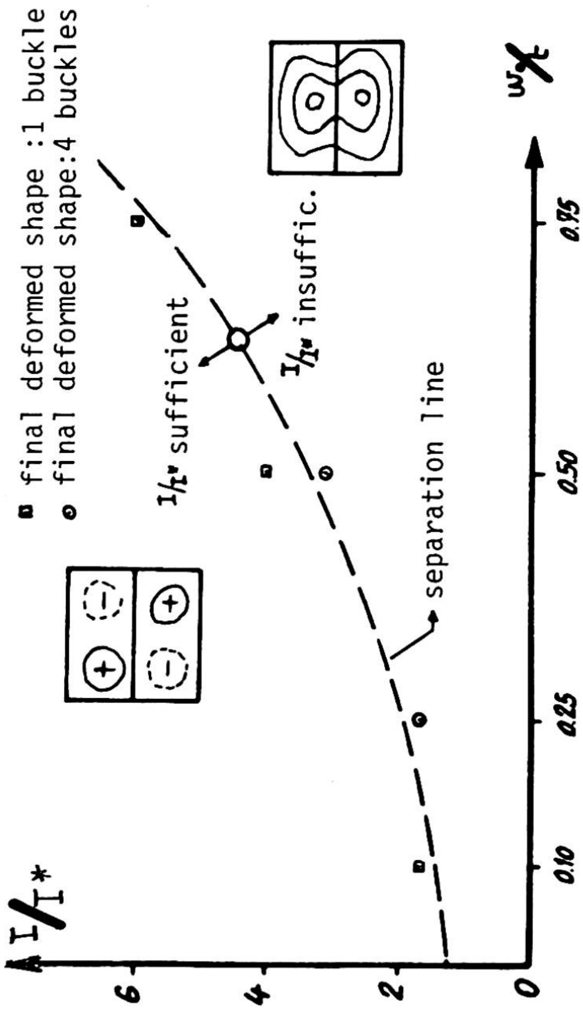

- Fig. 1 -

Professor P. DUBAS was the first to show, by experimenting on stiffened box girder models |D1|, that the $\gamma^*$ concept was especially dangerous in the case of compression panels reinforced by many stiffeners.
The same result had been obtained theoretically by SKALOUD and NOVOTNY |A6| in the case of one or two stiffeners. Very recently, a collaborator of P. DUBAS, B. ROUVE, has obtained more striking results by using |A13| a large displacement finite element technique (see figures 2 and 3).

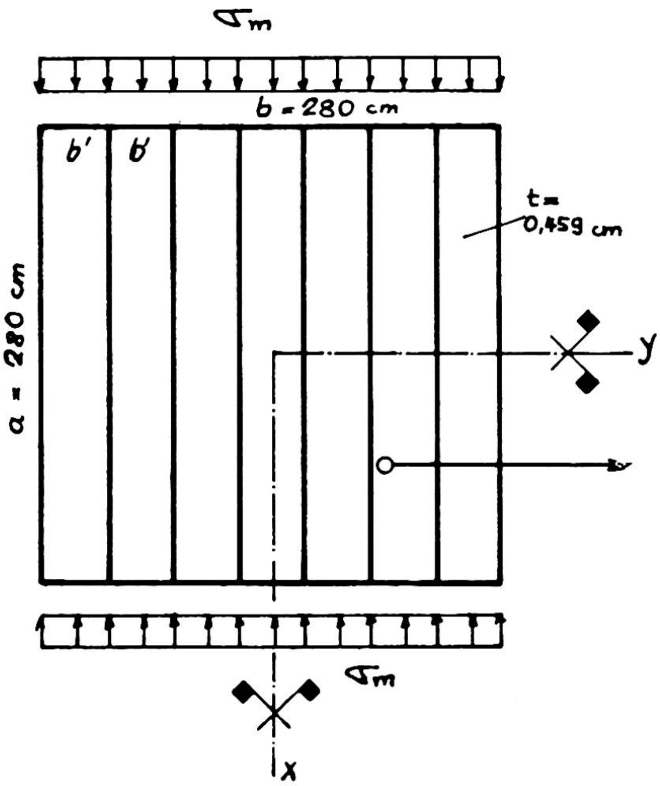
It begins to be possible, now, to derive the coefficient m arising in the relation

$$\gamma = m\,\gamma^*$$

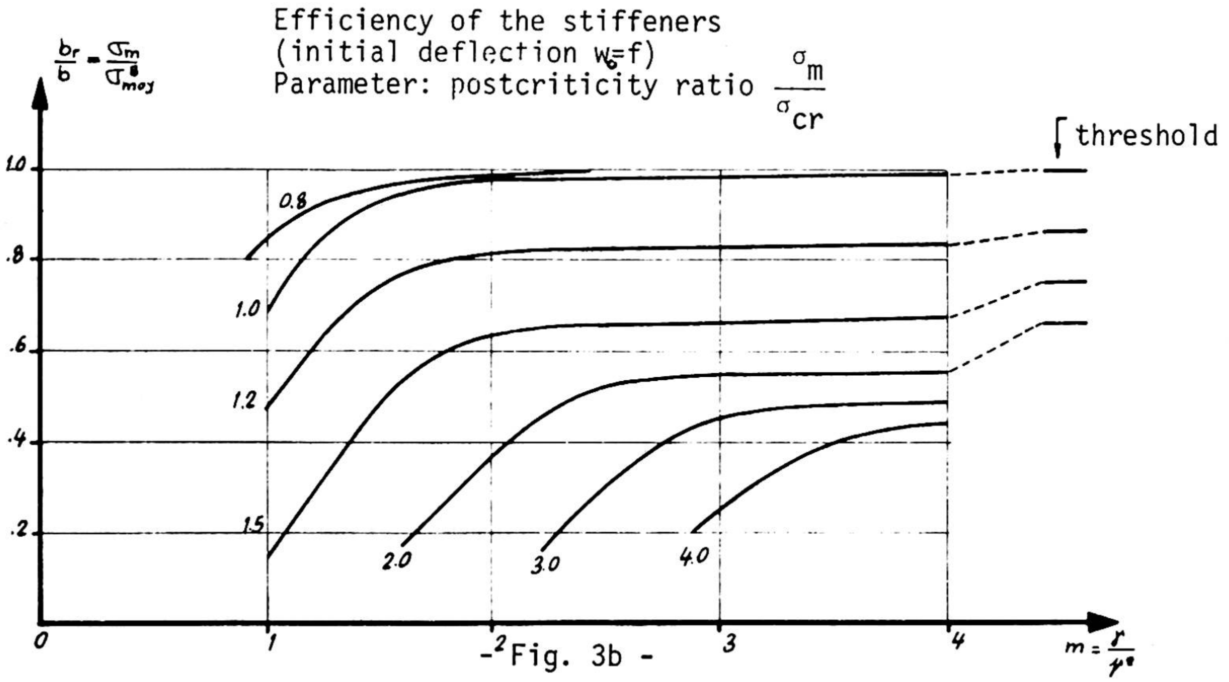
*theoretically*, as a function of the extent of the postbuckling domain in which the stiffeners are requested to remain quasi-straight, as well as of the initial imperfection of the panel considered.




- Fig. 2 -




- Fig. 3a -




- Fig. 3b -

This shows, in passing, the importance of a good quality of fabrication or, in other words, of close tolerances, not only on the geometrical imperfections, but also on the structural imperfections represented by the residual stresses. While some measurements on real structures have already been made |B10| and tolerances are given in many national specifications (e.g. |D.14|) no agreement could still be reached at the ECCS level within the subcommittee 8/3 (plate buckling) of ECCS. It is clear that tolerances very exacting and difficult to control, such as those proposed in the Merrison Rules |D4| can have a very deleterious effect on the price of fabrication and therefore hamper very much the development of steel bridges.

This is the more true because these rules impose the complete recording, in the factory, of the fabrication phase and a lot of corresponding measurements.

It is now generally acknowledged (|B3|,|B6|,|B7|,|B8|,|B9|) that the accidents that occurred under erection on four big box girders were due partly to poor detailing |B3|, and partly to inadequately low safety factors |D1||D2|, |B7|. Specialists differ about the relative importance of these two reasons. Personnally, I would say that, roughly, each of them bears half of the responsibility for the collapse.

It becomes clear, now, that the Merrison Rules, which had to be written in emergency conditions, are presently superseded; when they were published, they did represent a remarkable progress in many fields (shear lag, diaphragms,etc.). The engineering world was awaiting since two years the publication of a book that would explain clearly their scientific background. It seems, now, that this book will never appear.

It was clear, at the Conference held in July 1976 at Imperial College, London, that, due to the severity of these Rules, that are still in force in waiting for a new british code on steel bridges (the future BS 116 !), a very wide gap separates presently the fabricators and the research men who influence the design rules.

In the mind of present reporter, this is due to two very distinct reasons:

a) due to the advent of the limit state philosophy and therefore of the probabilistic interaction coefficients, the number of loading cases seems to increase to inacceptable limits. In London, Wex and Brown |D33| explained that, for a two-span continuous bridge, 1744 loading cases have in principle to be investigated. Even if more than 90 per cent of these cases may be immediately discarded on the basis of engineering judgment, it remains nevertheless that about ten different loading cases should normally be investigated ;

b) due to the introduction of the concept of *ultimate* strength limit concept, which replaces the concept of *elastic* response under *working* load, the analysis becomes now frequently elasto-plastic or, at least, non linear.

The duration and the price of the study of a box girder bridge are soaring up in such a way that some fabricators claimed in London for a dictator who would obtain the powers to simplify sufficiently the design rules.

To the present reporter, it is clear that the price to be optimized is the integrated price, comprising the study, and that therefore the length and difficulty of this study must be kept within reasonable limits.
It is equally clear that fabrication tolerances must be imposed, but they must be reasonable and simple to apply. The only way to reach this goal is to base them on extensive measurements on actual structures and to let fix them by a

commission composed, by equal parts, of research men, design specialists and fabrication and erection engineers, working hand in hand.

With these statements in mind, we shall now try to offer a general view of the present state of development of the ultimate design rules. It must be admitted that the research has been so intensive since 1970 that we are still in a very fluid state. The outcome of the studies undertaken in Western Germany is just beginning to appear and it seems that new-research will be launched in the U.S.A. under the sponsorship of the Subcommittee on Ultimate strength of box girders of the ASCE - AASHTO Task Committee on flexural members. Present report has therefore every chance to become obsolete in a foreseable future.

We shall first discuss plate girders and then, box girders. After that, we shall review some interim conventional design rules still based on the linear buckling theory. This report will conclude with brief considerations regarding the diaphragms of box girders, effect of local loads and other special problems.

## 3. ULTIMATE STRENGTH DESIGN OF PLATE GIRDERS.

Ultimate strength models for computing the ultimate carrying capacity of a stiffened panel in shear (plus eventual shear and compression) do not consider the linear plate buckling load as a limiting load, but as a basic value. Therefore, the design methods based on this concept are valid for technical construction with initial imperfections.

These methods all assume that the ultimate capacity can be evaluated from a cooperation of three independent actions, i.e.:
shear field-tension field-frame mechanism.
All methods assume that at ultimate strength, the shear field contributes the $\tau_{cr}$ stress of the linear theory, which this author feels to be a gratuitous assumption (see in this respect |C15|.) Some authors neglect completely the effect of the frame mechanism. The equivalent stress, calculated from the shear and the tension field, cannot exceed (under the factored loads !) the yield stress of the web material. When the frame mechanism is considered, one must take account of stresses induced by the bending moment of the girder by means of the bending rigidity and plastic moment of the flanges.

The basic model for ultimate shear strength of a plate panel is that developed between 1959 and 1961 at Lehigh University by K. BASLER (|C3|,|C4|, |C5|)under the guidance of B. THÜRLIMANN. This semi-diagonal tension field had already been studied by H. WAGNER in 1920 in the case of extremely thin webs (pure diagonal tension field |C2|) and it can be considered that a rather good approach of this basic model above was already given by RODE in 1916 |C1|.

The BASLER approach was criticized by several researchers(|C23|,|C6|,|C15|)on the ground of a computation error. This objection is not accepted by BASLER. Anyway, there are presently about ten different models for the ultimate strenth of a plate panel subjected to shear with eventual additional bending (|C8|,|C9| |C10|,|C11|,|C12|,|C13|,|C14|,|C16|,|C17|,|C20|,|C 21|,|C22|).

The corresponding theories,whose main characteristics are summarized in TABLE 1, are briefly described in the State of Art Report |C19| to which we are forced to refer, due to lack of space. These theories have various degrees of generality; the most general should be applicable to a plate panel

| | Aspects | Aarau | Cardiff | Göteborg | Karlsruhe | Lehigh | Osaka | Prague-Cardiff | Stockholm | Tokyo | Zürich |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Clarity of the method | fair | very good | very good | very good | good | fair | very good | very good | good | very good |
| 2 | Simplified method, easy for users | no | yes | yes | yes | no | no | no | yes | no | yes |
| 3 | Computer needed | no | charts | no | no | yes/no | yes | no/yes(1) | no | no | no |
| 4 | Account taken of longitudinal stiffeners | yes | yes | no | yes(2) | yes/no(3) | no | yes | no | no | no |
| 5 | Applicable to hybrid girders | yes | yes | yes | yes | yes | no | yes | yes | yes | yes |
| 6 | Applicable to unsymmetrical girders | yes | yes | yes | yes | yes | yes | no | no | yes | yes |
| 7 | Applicable to composite girders | no | nothing said | nothing said | yes | nothing said | no | no | no | nothing said | nothing said |
| 8 | Details for calculation of bending capacity | yes | yes | yes | yes | yes | no | yes | yes | yes | yes |
| 9 | Consideration of normal forces | no | no | yes | yes | no | no | yes | yes | no | yes |
| 10 | Details about $\sigma_{cr}$ of compression flange | yes | mentioned | no | mentioned | yes | no | mentioned | mentioned | no | no |
| 11 | Pure shear | yes | yes | yes | yes | yes | yes | yes | yes | yes | yes |
| 12 | Normal stresses due to bending | yes | yes | yes | yes | yes | no | yes | yes | yes | yes |
| 13 | Combined shear and bending | yes | yes | yes | yes | yes | no | yes | yes | yes | yes |

(1) for solving a cubic equation ;

(2) in the serviceability state ;

(3) computer method takes account of longitudinal stiffeners.

TABLE 1 - Summary of the aspects of the design methods.

dissymetrical (that is, with unequal steel flanges or with the flange replaced by a reinforced concrete deck), hybrid, reinforced by various transverse and longitudinal stiffeners and subjected simultaneously to internal resultants M, N, V (the N is important, e.g. for cable-stayed bridges). It can be safely said that none of the theories presently proposed does cover this whole field. On the other hand, because the problem is extremely complicated, these models are all based on different simplifying assumptions, so that someone has rightly remarked that there are as many models as members in the research committee. The author must mention, in addition, that, even if some theories are accompanied by simplifying charts obtained on computer and intended to design offices, it is difficult to avoid a "cooking recipe" type of presentation, with the corresponding danger of misunderstanding. At the last Colloquium of W.C. II of IABSE (Dresden, September 1975), the author has emphasized |A16| the danger that this type of rule may present for the profession. A clear physical insight of any design rules is absolutely necessary and it cannot be said, for example that the Merrison Rules |D4| are a model in this respect. Anyway, Working Group 8/3 of ECCS has undertaken the tedious task to calibrate the ten methods against *all* valid known tests. A similar calibration is contained in the American Guide about structural stability, edited by Bruce JOHNSTON |A19|.

The origin of the tests used is indicated below, while TABLE 2 gives, for each method, the mean value "m" of the ratio of predicted to observed load as well as the corresponding standard deviation.

Test results from different sources have been used to calibrate the design methods. These tests have been classified into two different groups :

(a) webs with transverse stiffeners only ;

(b) webs with transverse and longitudinal stiffeners.

Only the tests satisfying the requirements proper to each method have been considered for the statistical calculations (TABLE 2).

The sources are as follows :

BASLER, YEN, MÜLLER and THÜRLIMANN
ROCKEY and SKALOUD
LONGBOTTOM and HEYMAN
SAKAI, NISHINO and OKUMURA
EVANS, PORTER and ROCKEY
DIMITRI and OSTAPENKO
SCHUELLER and OSTAPENKO
SKALOUD
D'APICE and COOPER
COOPER, LEW and YEN
KONISHI
FUJII
CARSKADDAN
STEINHARDT and SCHRÖTER

| Method | Transverse stiffeners only | | | Transverse and longitudinal stiffeners | | |
|---|---|---|---|---|---|---|
| | n | m | sd | n | m | sd |
| Aarau | 42 | 0.99 | 0.20 | 48 | 1,01 | 0,21 |
| Cardiff | 44 | 1.02 | 0.06 | 66 | 1.00 | 0.07 |
| Göteborg | 33 | 0.99 | 0.12 | - | - | - |
| Karlsruhe | 78 | 0.97 | 0.14 | 102 | 0.96 | 0.14 |
| Lehigh | 48 | 1.02 | 0.12 | 60 | 1.02 | 0.11 |
| Osaka | 36 | 1.07 | 0.17 | - | - | - |
| Prague-Cardiff | 32 | 0.98 | 0.08 | 46 | 0.99 | 0.07 |
| Stockholm | 33 | 0.96 | 0.11 | - | - | - |
| Tokyo | 40 | 0.95 | 0.15 | - | - | - |
| Zürich | 30/26 | 0.79/1.06 | 0.25/0.18 | - | - | - |

TABLE 2 Calibration against test results.

n : number of tests compared with design method ;

m : mean value of ratio of predicted to observed load ;

sd: standard deviation.

It is seen that, if some simple methods are deliberately safe, all can be used with confidence at least in the case of static loading provided they are not applied to exceptional cases which would be completely outside the field of the tests.

## 4. ULTIMATE STRENGTH DESIGN OF BOX GIRDERS.

While everybody agrees that there is a certain interaction between the webs and the compressed flange of a box girder, the complexity of the problem is such that, presently, all methods apply to isolated stiffened flanges subjected to a compression force either constant (V=0) or variable along the span ( $V \neq 0$), plus eventually torsion ($M_t \neq 0$).

A problem still unsolved is whether it is economically interesting to "expand" the cross section in order to increase the strength of the girder, even at the expense of a reduction of the collapse stress, or not. A study of this problem would be especially welcome.

While computation models applicable to panels reinforced by very stocky stiffeners and used e.g. in naval architecture are known for a long time (see e.g. the state of the question made in |D2|), non linear models taking account of the actual support conditions of the plate panel (roughly simple support on the four edges), of the orthotropic character of the orthogonally stiffened panel and the membrane restraining stresses have appeared only very recently. We shall therefore divide the following discussion in two parts.

### 4.1. MODELS APPLICABLE TO STRONGLY STIFFENED PANELS, WHICH BUCKLE ALMOST CYLINDRICALLY.

After the YOSHIKI model |cf. D2| , the ultimate strength of stiffened panels could be evaluated by the FAULKNER formula, |cf. D2| , which, like the WINTER formula, is based on the concept of effective width and which was derived from a series of full-size buckling tests of stiffened panels of warships.

This type of model is especially cultivated in Great-Britain, where it seems to be postulated that optimum design corresponds always to very stocky stiffening.

The models used for the design of stiffened plates were developed on the basis of several previous valuable studies of the non linear behaviour and ultimate strength of isolated panels, first in the elastic domain |D20|, then in the elasto-plastic one, with account taken of an initial curvature and of residual stresses (|D6|, |D7|,|D21|,|D22|,|D25|).

The first of this models, presented in the Merrison rules |D4|,was considerably criticized as being complicated and oversafe for what regards the serviceability criterion. The subsequent models tried to correct this main defect. They were developed by DWIGHT and LITTLE at Cambridge |D6|, HORNE and NARAYANAN at Manchester |D8|, WALKER and MURRAY at Melbourne |D7| and they have benefited from a considerable amount of tests, some of them on stiffened panels discarded from the original Lower Yarra bridge structure. Dr. DOWLING and his collaborators at Imperial College have developed a more general model |D31|, which is said to be able to take account of postbuckling effects, but has not been tested in the field of flexible stiffeners up to now. The degree of agreement of all those models with the tests will be discussed further on .

## 4.2. NON LINEAR MODELS APPLICABLE TO THE LARGE DEFORMATIONS OF RECTANGULAR PANELS SIMPLY SUPPORTED ALONG ALL FOUR EDGES.

Prof. DUBAS was the first to demonstrate experimentally |D3| that designs of compressed stiffened panels based on the $\gamma^*$ star concept were highly dangerous and that the usual safety factor of 1.35, applicable to shear panels of plate girders, had to be considerably increased in the case of compression panels. MAQUOI, MASSONNET |A8| and SKALOUD |C7|showed that this conclusion could be derived theoretically in the case of stiffened imperfect panels reinforced by a unique longitudinal stiffener and uniformly compressed.

The Zürich design method proposed by Prof. DUBAS |D38| derives directly from his tests and is a modification of the classical (linear) buckling approach to stiffened plates.

As a measure of the stiffened plate's capacity to carry load beyond the critical buckling stress, use is made of MASSONNET's coefficient

$$m = \frac{\gamma_{post\text{-}cr}}{\gamma_{linear}}$$

where $\gamma = \frac{EI}{bD}$

is the well known relative rigidity of a stiffener.

DUBAS found by his tests that an empirical value of m = 4.5 gave a good agreement with the theoretically predicted failure loads. To estimate the moment carrying capacity of a given stiffened box girder, he proposes that :

1) The effective rigidity of the stiffeners be divided by five ;
2) the value of $\gamma$ so obtained should be used (using eventually the charts of |A1, 2| to calculate $\sigma_{cr}$ ;

3) the effective breadth of flange be calculated using $\sigma_{cr}$ and hence the elastic section properties as obtained ;
4) the maximum moment be obtained using these section properties and $\sigma_{max} = \sigma_r$ (yield stress).

The first accurate evaluation of the postbuckling strength was obtained in 1971 by MAQUOI and MASSONNET's theory |D2|, which was subsequently shown to be in good agreement with an important series of tests (|D5|,|D9|,|D10|).

The theory is based on a suitable generalization of the von KARMAN-MARGUERRE non linear equations of moderate deflections of plates to orthotropic, eccentrically stiffened plates. The main assumptions are

a) The initial imperfection is a doubly sinusoïdal buckle (fig. 4)
b) The additional deflection is affine to the first buckling mode of the linear theory, which is also a doubly sinusoïdal buckle.
c) The boundary conditions of a plate panel situated between two successive nodal lines are those indicated in fig. 4.

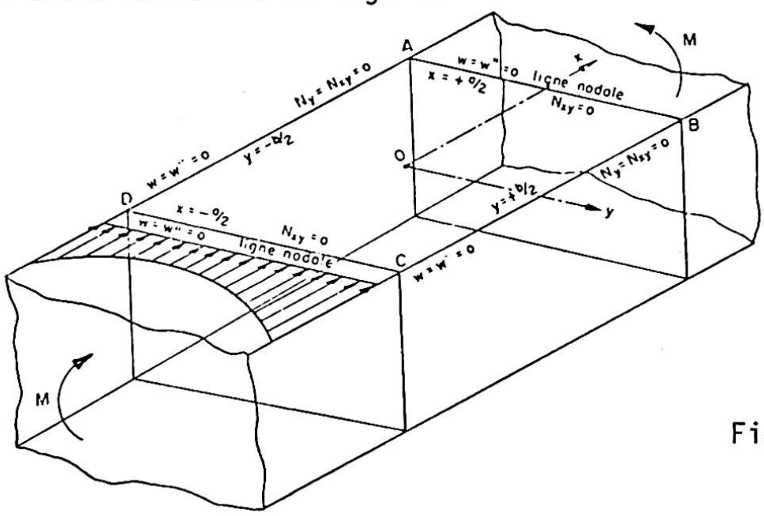
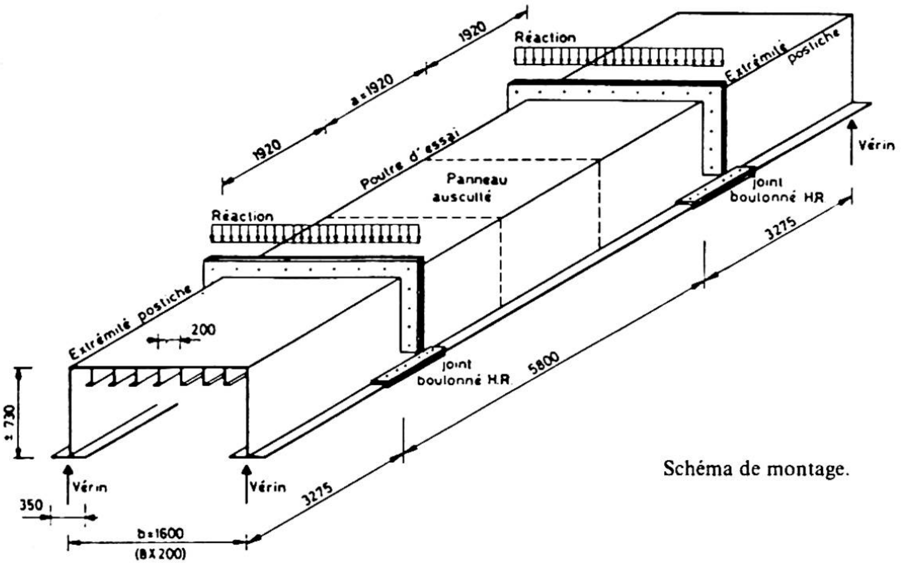



Figure 4

In the equations, the stiffeners are assumed to be "smeared" continuously on the compressed plate. The theory gives the efficiency $\rho_g$ of such a "continuously stiffened plate". To take account of the discontinuous character of the stiffener, the effective width formula of FAULKNER is applied, which gives a local efficiency $\rho_l$. The total efficiency $\rho_t$ is assumed to be the product of the two efficiencies $\rho_g$ and $\rho_l$ above.

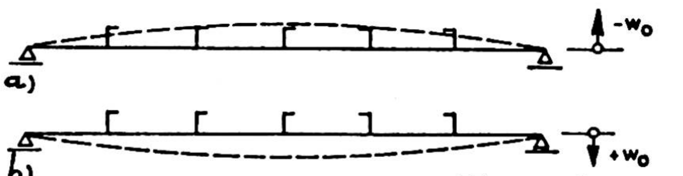
The ultimate strength is assumed to be reached (in agreement with the recommendations of WOLMIR |A19|) when the mean value of the membrane stresses along the unloaded edges AD or BC of the compressed panel, reach the yield point. A series of large size models |D10| (Fig. 5) have been executed which have shown that

a) above criterion of collapse was accurate, but a little safe ;
b) the theory was in good agreement with the tests.
c) there was a considerable postbuckling strength reserve in the case of slender stiffeners.




- Figure 5 -

The Karlsruhe method developed more recently by STEINHARDT, VALTINAT and RUBIN, (|D17|,|D18|,|D32|) has several characteristics in common with the Liège method. However, it is more accurate, because it takes account of the discontinuous character of the stiffeners and leaves open the form of the transverse section of the buckle, which is determined so as to minimize the potential energy. It also distinguishes two types of collapse, by bending on the side of the stiffeners (fig. 6.a) and by bending on the side of the compressed plate (fig. 6.b).




-Figure 6 -

In addition, the thesis by Dr. RUBIN |D18| presents a series of charts that should facilitate considerably the practical use of the method.

A useful generalization of both non linear theories above has been presented in 1976 as Doktor Ingenieur Thesis by Dr.CHANG SUNG Pil |D19|. It is a non linear theory, based almost on the same assumptions as the Liège and Karlsruhe method. The ultimate strength N° 1 is the load for which the compression stress at the edge of the most stressed stiffener reaches the yield point. The ultimate strength N°2 is that for which, in the middle plate of the plane, the equivalent stress reaches the yield stress.

The main progress of the thesis is to be applicable also to closed section profiles. For open section stiffeners, it becomes almost identical to the MAQUOI-MASSONNET theory.

Large closed section stiffeners are now used quite generally for the orthotropic bridge floors. In his paper to the London Conference, DUBAS |D34| demonstrated that they furnish also a convenient and economical solution for longitudinal web and plate stiffeners. Intersection problems with the transverse stiffeners can be avoided by placing each category of stiffener on a different side of the plate. In his discussion of DUBAS's paper, KLEMENT (Austria) recommended highly the closed section stiffeners.
They are used in bridges in Austria since 20 to 30 years with no corrosion problems.

## 4.3. CHARACTERISTICS OF THE VARIOUS METHODS.

The degree of generality of the various methods may be seen from TABLE 3 in which the available methods have been separated in the two categories corresponding to sections 4.1. and 4.2. above, respectively.

| Aspects of Stiffened Compression Flange, Covered in the Different Methods | Method | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Merrison Rules | Cambridge Method | Monash Method | Manchester Method | Imperial College | Zürich Method | Liège Method | Karlsruhe Method |
| 1. Asymmetry of cross-section of flange about horizontal axis | √ | √ | √ | √ | √ | - | - | √ |
| 2. Buckling of plate between stiffeners | √ | √ | √ | √ | √ | √ | √ | √ |
| 3. Torsional buckling of stiffener outstand | √ | - | - | - | √ | - | - | - |
| 4. In-plane transverse stresses in flange plate | √ | - | - | - | - | - | - | - |
| 5. In-plane shear stresses in flange plate | √ | √ | - | - | √ | - | - | - |
| 6. Locally applied lateral loading | √ | - | - | - | √ | - | - | - |
| 7. Variation in axial load along length | √ | √ | - | - | √ | - | √ | - |
| 8. Overall curvature of box girder | √ | - | - | - | √ | - | - | - |
| 9. Overall postbuckling behaviour of flange | √ | - | - | - | ? √ | - | √ | √ |
| 10. Welding residual stresses | √ | √ | - | √ | √ | - | - | - |

TABLE 3

## 4.4. AGREEMENT BETWEEN THEORY OF TESTS.

The five british and australian methods are in satisfactory agreement with the corresponding tests, in which the lateral edges were kept free. To this reporter, these are wide column tests rather that tests on actual stiffened com-

pressed flanges, which are obviously held laterally (w=0) by the webs of the box girder.
For the details of the statistical comparison, which take two pages, the reader is referred to |D38|.

We shall reproduce here only the TABLE 4, which gives the statistical comparison between the 8 design methods and 12 actual tests on complete box girders, undertaken partly in Liège and partly in London. It is seen that all methods are safe - with varying degrees of safety - and give decent standard deviations, the more dispersed being the Karlsruhe and Monash Methods.

## 4.5. SHEAR LAG.

A problem that complicates the design of a box girder which is subjected to a bending moment M(x) variable along the span and accompanied therefore by the shear force $V = dM/dx$ is that of shear lag. This expression designates the unequal distribution of the normal stresses over a cross section that is due to the (first order) in plane deformations of the component plates due to in-plane shear. This phenomenon was studied in the case of perfectly flat panels by ABDEL-SAYED |D13|. This problem was carefully studied at Imperial College by DOWLING and his collaborators, working for the Merrison Committee (|D11|,|D12|) and simple approximations for design have been obtained. They are more refined than the rules given by the german specifications about steel bridges |D14| because they take account of the orthotropic character of the compressed flange, or, otherwise speaking, of the effect on shear lag of the longitudinal stiffeners.

In the postbuckling range, there is another type of shear lag; MAQUOI and MASSONNET have shown (|D15|,|D27|) that, when the two types of shear lag occur simultaneously, the net effect is less than the product of the two isolated effects. In other words, a safe value of the efficiency of the girder is obtained by multiplying the efficiency $\rho_t$ as given by the MAQUOI-MASSONNET theory in the case of pure bending ($V = 0$) by the efficiency factor $\rho_o$ corresponding to the first-order shear lag of the perfectly flat panel.

In a very recent paper presented at the July 1976 london Conference, DOWLING, MOOLANI and FRIEZE, on the basis on three point-loaded box girder models |D28|, show that, in the ultimate condition, shear lag has little weakening effect on flange strength, due to plastic redistribution, and may be completely ignored if :

a) the stiffeners are stocky enough to sustain a strain equal to 2.5 the times yield strain without significantly unloading ;

b) the plate panels are capable of being strained up to 4 times the yield strain without significantly unloading.

The shear lag effects are also being studied now in Germany |D30|, but the researches are not enough advanced to produce definite conclusions.

The paper by LALLY and WOLCHUK |D29| gives interesting details about the american specifications for box girders bridges. It indicates that the design of plating in compression is based on an elastic-plastic buckling curve, which, like in Great-Britain, disregards any postbuckling effect.

Two still rather obscure problems are :
a) the design of the stiffeners for sufficient ultimate strength |D35| and
b) the behaviour of the compressed plate when (due to torsion and even shear-lag), it is subject to combined direct and shear in-plane loading |D36|.

| DESIGN METHOD | | TEST RESULTS LIEGE 6 tests | TEST RESULTS IMPERIAL COLLEGE 6 tests | TOTAL 12 | N |
|---|---|---|---|---|---|
| ZURICH | n | 6 | 6 | 12 | 5 |
| | m | 0.869 | 1.004 | 0.937 | |
| | sd | 0.089 | 0.052 | 0.099 | |
| MERRISON | n | 6 | 6 | 12 | 1 |
| | m | 0.702 | 0.844 | 0.773 | |
| | sd | 0.058 | 0.138 | 0.126 | |
| LIEGE | n | 6 | 6 | 12 | 0 |
| | m | 0.751 | 0.878 | 0.815 | |
| | sd | 0.108 | 0.076 | 0.113 | |
| CAMBRIDGE | n | 6 | 6 | 12 | 0 |
| | m | 0.704 | 0.804 | 0.754 | |
| | sd | 0.063 | 0.048 | 0.075 | |
| KARLSRUHE | n | 6 | 6 | 12 | 1 |
| | m | 0.817 | 0.915 | 0.866 | |
| | sd | 0.107 | 0.178 | 0.170 | |
| MONASH | n | 6 | 6 | 12 | 0 |
| | m | 0.693 | 0.762 | 0.729 | |
| | sd | 0.168 | 0.110 | 0.100 | |
| MANCHESTER | n | 6 | 6 | 12 | 0 |
| | m | 0.756 | 0.871 | 0.814 | |
| | sd | 0.096 | 0.088 | 0.108 | |
| IMPERIAL COLLEGE | n | 6 | 6 | 12 | 0 |
| | m | 0.693 | 0.758 | 0.726 | |
| | sd | 0.086 | 0.123 | 0.111 | |

n = N° of tests compared with design method
m = Mean value of ratio of predicted to observed load
sd = Standard deviation
N = N° of cases where ratio of predicted to observed load exceeds one.

TABLE 4 - Calibration against box girder tests.

## 5. DIAPHRAGMS.

Not so long ago, diaphragms of box girders were verified by very crude methods. Because of the collapse at Milford Haven, which is attributed to a diaphragm weakness |B5|and due to the capability of elastic or elastoplastic finite elements, including eventually large displacements, a lot of studies have been devoted to diaphragms in Great-Britain. The first results, applicable to thick unstiffened diaphragms, are given in the Merrison Rules |B1| and studied in detail in |E2|. More recent results on stiffened diaphragms of trapezoïdal box girders have been published |E3| or presented in July 1976 at the London Conference (|E4| |E5|).

## 6. FABRICATION PROBLEMS.

The discussion at the London 1976 Conference have shown that it was absolutely necessary to establish a set of realistic fabrication tolerances that would be accepted together by the designers and fabricators. This would probably require an international cooperation.

We shall content ourselves, here, to mention the paper by HORNE and NARAYANAN |D24| in which the authors describe the effect on ultimate strength of welded stiffened panels loaded in longitudinal compression of various fabrication procedures (intermittent versus continuous plate/stiffener welds), various types of local imperfections, various procedures for restoring longitudinal straightness in panels which were bowed during fabrication. The perhaps most astonishing result pertains to the eccentric longitudinal stiffener splices on flat stiffeners, which had been heavily blamed after the Melbourne disaster (see |B6| to |B9|).

According to |D24|, these eccentric splices should be equivalent to concentric splices for what regards ultimate strength, provided a continuous connection exists between stiffener and plate.

## 7. OTHER PROBLEMS OF PLATE GIRDERS : PLATE GIRDERS WITHOUT STIFFENERS, EFFECTS OF CONCENTRATED LOADS, HYBRID GIRDERS, SPECIAL PROBLEMS.

There are a lot of special problems concerning plate and box girders which should be now discussed. The first of them concerns the effect of more or less concentrated transverse loads applied to the compression flange and acting on the girder in combination with the general resultants M, V, N. This problem seems to-day almost clarified, due to the efforts of ROCKEY |C7||F4| and BERGFELT |F5|. The swedish school of research has also proposed, since about 15 years, a set of rules for the design of plate girders of moderate size, used in buildings, and subject to distributed static loads. These girders have no stiffeners, except at the supports. The main progresses in this field have been obtained by GRANHOLM, BERGFELT |F1|and HÖGLUND |F2| . HÖGLUND has also given rules |F3| to take account of the presence of holes in the web.
The swedish rules are based on the postbuckling resistance of the thin web. Commission 11 of ECCS, under the chairmanship of Prof. REINITZHUBER, has actively studied the problem and ECCS Recommendations for the design of this type of girder will be published very soon.

At the London Conference of July 1976, van DOUWEN has demonstrated |F6| the technical and commercial advantages of this type of girders, which enable the fabricator to reduce considerably his investment in plates compared to the considerable stock which must be maintained in the case of rolled profiles.

## 8. THE PRO- AND AGAINST ULTIMATE STRENGTH DESIGN RULES FOR PLATE AND BOX GIRDERS.

While ultimate strength design rules are in principle vastly superior to the conventional rules based on the linear theory of buckling, there are numerous objections against them, which eventually carry a lot of weight.

### 8.1.

In principle, - as was emphasized in the London discussions at ICSPS by professor STEINHARDT, structures subject to moving loads should be designed elastically (even if advantage is taken of non linear membrane effects) and real plastic design should be reserved to compact sections, in the AISC Specifications meaning of this word. The author has always defended this point of view; indeed, not to speak of the fatigue problems, there seems to be a blunt contradiction between designing sections for ultimate strength and still continuing the determine the M,N,V diagrams on continuous girders by using elastic theory; the fact that a similar situation exists in reinforced and prestressed structures is not sufficient to justify it.

### 8.2.

Above limitation of ultimate strength methods to bridges can be obtained automatically if proper control is made of the serviceability limit state. We think, here, of the "breathing" of webs, which can produce fatigue cracks in the heat-affected zones near the flanges and/or transverse stiffeners (|A11|, |A12|).

It seems that our tchecoslovakian colleagues have similar ideas, because the paper by DJUBEK and SKALOUD |C18| about the new tchecoslovakian standard for plate girders distinguishes carefully between fixed and moving loads. In the case of moving loads, non linear elastic rules, due to DJUBEK, are proposed, while plastic mechanisms (ROCKEY-SKALOUD) are reserved for the case of fixed loads. According to DJUBEK, Professor BROUDE has developed similar rules in USSR for the design under moving loads.

The ultimate concession regarding plastic action that could be made to ultimate strength methods would be to take as ultimate load the shake-down load (in the sense of this expression in plastic theory) and not the collapse load. Unfortunately, this would still complicate further the design rules.

### 8.3.

The elasto-plastic phenomena at collapse being extremely intricate, the ultimate strength methods are forced to rely on simplified models and assumptions. While at least some of these models are conceptually simple, the design rules take easily the shape of "cooking recipes", that the designer does not understand, what the present author considers to be productive of misunderstanding and therefore highly dangerous for the profession, not to speak of the unpleasantness of the teaching |A16|.

8.4.

The ultimate strength methods cannot be general. New technical developments about cable-stayed bridges, (biaxial compression plus shear in the foot of the towers, etc) offshore structures, etc... are continuously bringing about new problems that must be solved without waiting until the proper corresponding ultimate strength model is developed.

8.5.

For all these reasons, that have been analyzed a little differently by SCHEER and NÖLKE |A18| at the London Conference on Plated Structures, there is a need for interim design rules, still based on the linear buckling theory, because the designers need them to build.

These rules |A16| have been developed in cooperation between the german Working Group "Plate Buckling" of the german "Unterausschuss für Stabilität" and the Working Group 8/3 of ECCS, and particularly Prof. DUBAS, Dr. MAQUOI and the author. They have appeared as an Appendix to the ECCS Recommendations for the Design of Steel Structures and are justified scientifically in the Manual on Structural Stability |A15|.

For the detailing problems (welded or bolted connections of plates and stiffeners), above rules give detailed prescriptions which should prevent the return of collapses during erection.

The safety has been adjusted by the introduction of correction factors so as to take account of the varying amount of postbuckling strength with the type of loading of the panel (shear or compression) as demonstrated by the tests. While it is openly admitted that this "manipulation" of the safety factors is not a very scientific procedure and that the safety should rather be a result of the basic principles of the design theory, it is felt that the resulting rules have the considerable advantage of being both simple and safe. The author believes therefore that, in spite of their title "provisional rules", they will still be applied for a long time.

## 9. RECOMMENDATIONS AND CONCLUSIONS.

9.1.

An extremely large research effort has been made these last six years towards a better understanding of the ultimate carrying capacity of plate and box girders, but it remains to digest the corresponding research papers in order to transform them into acceptable design rules.

9.2.

Quite apart from the particular problem of plate and box girders, the semi-probabilistic theory of limit states should be carefully weighted in order to maintain the computational effort of the designer within a decent percentage of the total price of the structure. For this to be obtained, methods should be found to identify the few limit states that are really governing.

9.3.

The Merrison Rules were interim rules, written in a period of emergency. They have performed their main duty, which was to protect against new accidents. The research launched by the Merrison Committee has brought a wealth of new scientific information.

9.4.

The Merrison Rules must now be replaced, as soon as possible, by more simple specifications, because they are hampering the development of long span steel bridges. The truth is somewhat in the middle between the Merrison Rules and the dangerous oversimplified rules that some designers would favour at any cost. It will, however, be difficult to obtain these desirable balanced rules by Committee work, because research men and fabricators have widely different views. Perhaps will it be necessary to appoint a kind of wise and competent "dictator" to "distill" the enormous amount of research now available.

9.5.

A set of realistic and easy to control tolerances should be established within a committee comprising by equal parts research men, high format designers and fabricators-erectors. IABSE and ECCS should be helpful in discussing this problem at the european or even worldwide level. The control of these tolerances could be restricted to a certain sampling (5 % ?) to be agreed upon by the parties. Anyway, continuous recording of measurements on all parts of a bridge at the factory during fabrication is inacceptable.

9.6.

Studies should be continued to investigate whether elastic shear lag (and possibly also stresses arising from restraint of warping are "eroded" by plastic action. These studies should comprise fatigue - or, at least, repeated loading - tests, in order to assess to which extent the conclusions apply to bridges.

9.7.

Systematic comparative computations should be undertaken to assess whether optimization requests the stocky stiffeners commonly used (especially in Great-Britain) or whether it is economical to "expand" the cross section, as explained in detail in the beginning of section 4.

9.8.

It would be extremely instructive to compare on a wide basis these realistic ultimate strength design rules that we are still awaiting with the provisional rules based on the conventional theory evoked in section 8.5., in order to see whether the increase in scientific accuracy is justified by a sufficient increase in safety and or by an economy of the total price of the structure (that is the price including the design expenses).

9.9.

Summarizing, the main question mark still is: To which extent are we entitled to take benefit of plastic redistribution in the case of bridges supporting moving loads ?

## 10. ACKNOWLEDGMENTS.

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SUMMARY

After a brief summary of the tendency of research these last years, which is towards the development of ultimate strength, the author examines critically the linear buckling theory and emphasizes its shortcomings. Then, he reviews briefly the ultimate strength models presented for plate and box girders and compares their prediction with the full set of available experiments. After a brief review of the problems concerning shear lag, diaphragms and fabrication problems, he discusses the pro- and against ultimate design rules and shows the interest of previsional rules still based on the linear theory suitably adapted.

RESUME

L'auteur présente un bref résumé de l'évolution de la recherche au cours des dernières années, qui tend vers le développement des théories de la résistance à la rupture, puis il examine de façon critique la théorie linéaire du voilement et souligne ses faiblesses. Il rappelle brièvement les modèles de résistance à la rupture utilisés pour les poutres à âme pleine et en caisson et compare leurs résultats avec l'ensemble des expériences réalisées. Après un bref résumé des problèmes de "shear lag", de "diaphragme" et de fabrication, l'auteur considère les avantages et les inconvénients des règles de dimensionnement basées sur la résistance à la rupture; il souligne l'intérêt de règles provisoires basées sur la théorie linéaire adaptée de façon adéquate.

ZUSAMMENFASSUNG

Nach einer kurzen Darstellung der Forschungsentwicklung der letzten Jahre, die in Richtung von Traglasttheorien tendiert, unterzieht der Autor die lineare Beultheorie einer kritischen Prüfung und unterstreicht ihre Mängel. Traglastmodelle für Vollwand- und Kastenträger werden besprochen und die entsprechenden Ergebnisse mit allen vorliegenden Versuchen verglichen. Nach einer kurzen Zusammenfassung der Probleme der verminderten Mitwirkung infolge Schubverzehrung, der Querscheiben und der Anfertigung erörtert der Autor die Vor- und Nachteile von Traglast-Bemessungsvorschriften und stellt fest, dass provisorische Vorschriften, die auf einer entsprechend angepassten linearen Beultheorie beruhen, von Interesse sind.