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Tolerances in Steel Plated Structures

Tolérances dans les structures en acier formées de tôles

Toleranzen in aus Stahlblechen geformten Tragwerken

by a Task Group under the Chairmanship of

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SUMMARY

Geometric and structural imperfections have an effect on the ultimate strength of steel plated structures. The national codes differ rather significantly in this respect. A statistical analysis is made of about 15,000 initial deflection measurements on full size bridges, performed in four european countries. Parallel theoretical considerations and a critical review of the few theoretical papers devoted to the study of the effect of imperfections enable the Task Group to propose definite values for the acceptable geometrical imperfections. It is also shown that, for practical purposes, no measurement of residual stresses should be imposed.

RÉSUMÉ

Les imperfections géométriques et structurales influencent la résistance ultime de structures en acier formées de tôles. A ce point de vue les codes nationaux montrent des différences importantes. Sur la base d'une analyse statistique d'environ 15 000 mesures de déformations initiales effectuées sur des ponts dans 4 pays européens, et d'une revue critique des quelques mémoires théoriques consacrés à l'effet des imperfections, le Groupe de Travail propose des valeurs définies pour les imperfections géométriques acceptables. On montre également qu'en pratique il n'est pas nécessaire d'imposer la mesure des contraintes résiduelles.

ZUSAMMENFASSUNG

Geometrische und strukturelle Massabweichungen beeinflussen den Bruchwiderstand von aus Stahlblechen zusammengeseztzten Tragwerken. Nationale Normen zeigen bemerkenswerte Unterschiede in der Erfassung dieser Einflüsse. 15 000 Messungen von initialen Verformungen wurden an Brücken in vier europäischen Ländern durchgeführt und statistisch ausgewertet. Theoretische Betrachtungen sowie die kritische Durchsicht der wenigen theoretischen Abhandlungen über den Einfluss dieser Massabweichungen erlauben der Arbeitsgruppe, konkrete Werte für geometrische Toleranzen vorzuschlagen. Es wird auch gezeigt, dass in praktischer Hinsicht eine Messung von Eigenspannungen nicht nötig ist.

1. INTRODUCTION

1.1.

In September 1976, at the tenth Congress of IABSE in Tokyo, the Chairman of present Task Group concluded his general report entitled "Progress in the Design of Steel Plate - and Box Girders" by some recommendations, two of them will reproduced hereunder [1].

"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 fabrication is inacceptable".

The chairman took the occasion of the gathering, in Tokyo, of many of the world specialists in the design and fabrication of big stiffened plated bridges to establish first contacts with several of them, in view to set up the Committee recommended in point 9.5. above, Committee that hereforth will be called the IABSE Task Group: T.G. "Tolerances in Steel Plated Structures".
Prof. BERGFELT (Sweden), Dr. W.C. BROWN (U.K.), Dr. C. CARLSEN (Norway), Dr. S. CHATTERJEE (U.K.), Prof. F. CIOLINA (France), Prof. P. DOWLING (U.K.), Prof. P. DUBAS (Switzerland), Prof. P.J. DWIGHT (U.K.), Mr.JACKSON DURKEE (U.S.A) Prof. L. FINZI (Italy), Prof. H. GACHON (France), Prof. C.P. HEINS (U.S.A.), Prof. KLEMENT (Austria), Dr. D.E. LEBEK (W. Germany), Dr.H. NÖLKE (W. Germany), Prof. K.C. ROCKEY (U.K.), Prof. J. SCHEER (W. Germany), Prof. M. SKALOUD (Czechoslovakia) and Mr. R. WOLCHUK (U.S.A.), agreed to work in this T.G. (solely by correspondence). Several informations were received from these members.

The chairman then obtained assistance of Mr. J. JANSS, Chief Engineer at the Belgian Research Centre of the Industry of Metal Fabrications (C.R.I.F.) as secretary of this T.G.

More recently, Professor H. ŠERTLER, of the University of Transport Engineering of JELINA (Czechoslovakia) stayed in the Department of the chairman in Liège and accepted to participate to the redaction of the Draft of the Task Group Report.

A paper written by Ch. MASSONNET and J. JANSS, entitled "Geometric Rolling and Workmanship Imperfections of Steel Bridge Elements and their Effects on their Ultimate Strength, with Emphasis on Plated Structures" [2], prepared at the invitation of the organizers of the Cardiff September 1978 Conference on "the New Code for the Design and Construction of steel Bridges" (that was cancelled due to delay in the completion of this British Standard), was distributed to all members in October 1978 as a first progress report of the Task Group.

The aim of this Task Group was defined more precisely by Dr. W.C. BROWN [3] on January 1977, at the request of the Bureau of the Task Group:
The main difficulty in the assessment of Tolerances in Plated Structures is to

find a general method for establishing the effect of imperfections of various kinds and for deciding which are the optimal limit imperfections. Indeed, it is completely valueless to collect the tolerance values presently in force in the leading countries and to try to compute, say, their arithmetic mean; the scientific value of such a mean is obviously completely unknown. It is valueless as well to print all the values of geometrical and material imperfections measured in various bridges and even to compute the basic parameters of their probability distribution (namely, in the case of a normal LAPLACE-GAUSS distribution, their mean value and standard deviation) without knowing the correlation between these imperfections and the loss of strength of the structures.

Paraphrasing the definition given by the late professor E. TORROJA of what is an "optimum structure", we could say that the *"optimum set of tolerances" is that set which minimizes the total cost of a definite structure, required to sustain definite sets of loads and to respect the stress, displacement, stability, corrosion, etc... constraints imposed by the Code* .

This definition should be understood to apply within the *semi-probabilistic theory of limit-states,* which has now been adopted everywhere in the world (CEB - ECCS, theory of limit states put forward in USSR as early as 1950, etc..). Therefore, in the particular case of steel bridges, the optimum set of tolerances will obviously depend on the definition of "actions" on a bridge, what is precisely a matter of considerable discussion nowadays.

1.2. Bases for the assessment of tolerances in steel plated structures.

The loading on the structure being supposed as known, we must know the response of the structure to that loading and verify whether this response meets the requirements of the structural code.

The strength of the structure being affected by its imperfections, the strength model recommended by the Code should be capable of taking into account the influence of tolerances adopted in construction.
Thus, making a comparison between the response and capability of the structure, the effect of imperfections on the safety can be assessed in a quantitative manner as required.

In practice, neither the load, response or capability of the structure is deterministic, since variations occur in the loading and in the material and geometric properties of the structures, which subsequently affect both the response and capability of this latter.

To enable the effect of these variations on the safety and reliability of the structure to be studied, we should develop a RELIABILITY ASSESSMENT SYSTEM for Plated Structures, that should be very similar in its structure to the Reliability Assessment System developed at LLOYD's Register [4].

The main purpose of the reliability system is to determine the effects of variations around a mean value in the parameters which describe the load, response and capability of a structure. If the mathematical model developed to represent the structure includes parameters which describe the imperfections, then, by varying those parameters, the effect of changing the tolerances associated with those imperfections can be studied.

1.3. Control of distortion and the additional cost involved.

Apart from its deleterious effect on the ultimate strength, the distortion due to the shrinkage of welds is not only unsightly in the eye of the beholder, but may be positively dangerous in the amount of hidden residual stress which it carries. It is not generally realised how much cost is incurred in the elimination or partial elimination of distortion, as much of this cost tends to be buried or absorbed into the natural cost of the structure being fabricated. It is, of course, quite a considerable sum, but J.D. THOMPSON reports [5]that, from an enquiry at a number of companies as to what average costs amounted to, he was amazed to hear very vague answers varying from "almost nothing" to "50 pounds per ton", which are equally bad. The general opinion of the Task Group is that it is most difficult, for those not directly involved in the fabrication industry, to exert any significant influence on industry practices, or even to determine the cost of prospective changes in practice. Accordingly, the members of the Task Group believe that it is more adequate to adapt the new design methods to the present state of development of good fabrication, so as to provide the required degree of safety, than doing the reverse, namely to impose stringent and costly controls for an any unquantifiable benefit in strength.

Anyway, because the Task Group is of the opinion that it should not be imposed to measure the residual stresses due to fabrication and assembling, it believes that, as a counterpart, the fabricator must follow a general code of Recommendations for efficient fabrication and good practice. These recommendations are very briefly [5]:

1) Efficient design in balancing welds about neutral axes ;
2) Avoidance of excessive use of weld metal. This applies both to number and size of welds ;
3) Ensure that fit up is as perfect as can be achieved - generally, it is less costly to produce a work well done than a bad work ;
4) Use appropriate welding procedures and sequences, noting how and to what extent the work distorts as welding proceeds. Generally, automatic or semi-automatic welding yields better results than manual welding.

The very few theoretical analyses (discussed in section 4) correlating the imperfections of plated structures with their ultimate strength are based on the assumption that the distortion of plate elements and residual stresses affecting them are those produced by conventional fabrication techniques. This opens the question of the acceptability of methods to reduce geometric imperfections following fabrication. The opinions of the Task Group members seem to diverge on this point.

In Belgium, it is generally forbidden to apply any type of procedure of above kind, because they introduce uncontrollable residual stresses and because no evidence exists that the ultimate strength has been improved.
However, the opinion in the United States seems to be different, as reflected by the following statement expressed by Mr. JACKSON DURKEE in his letter [6b]. "Your position taken (in Section 4.3. of the first Progress Report [2]) against attempts to reduce geometric imperfections following fabrication is not entirely valid, in my opinion. It is true, as you point out, that such attempts augment residual stresses. However, I wish to suggest that occasional straightening of fabricated bridgework sections is considered acceptable shop practice in the U.S., since the alternative would be re-fabrication along with scheduling delays - both of which are generally not justifiable under the usual circumstances. A certain amount of carefully controlled heat-straighthening of stiffened platework is considered an acceptable part of bridgework fabrication for those occasional sections that do not meet established tolerances or are otherwise believed to be unacceptable. There is no compelling reason to believe that residual-stress and ultimate-strength conditions are thereby rendered significantly worse than those of "unstraightened" welded platework".

On the other hand, the german code, DASt-Richtlinie 012, October 1978 [C.3], states in section 13.4., page 11, that :

"If tolerances values are exceeded, there has to be decided in mutual agreement with the agencies responsible for structural safety (Bauaufsicht) whether straightening or other remedies are required".
Besides the official rules set forth in section 13.4. of DASt-Richtlinie 012, there seems to be in practice an unofficial mutual agreement between all parties concerned about the following points :
- geometrical imperfections within generally accepted limits should not be made a reason for straightening procedures.
- Where heat straightening is unavoidable and seemingly advantageous (it should always be restricted to rare occasions only), such techniques should be used very carefully and only be done by skilled experts with the unanimous approval of all parties concerned.

1.4. Limits of validity of present survey.

All tolerances will be discussed in this document solely under the consideration of strength. Tolerances as governed by strength only may not necessarily provide an adequate standard of workmanship e.g. matching of connecting parts, appearance, etc...
For those purposes each fabricator should choose his own standard enabling him to produce a finished structure according to (design) drawings.

2. CODES.

2.1. Introduction.

This section contains the fabrication tolerances for the steel stiffened plates of plated bridges given by some leading Codes which were available to the members of the Task Group. Only the requirements for the geometrical tolerances of plate panels, longitudinal and transversal stiffeners, cross-frames and cross girders are considered here. The main data of each code are reproduced in a comprehensive table.

In the commentary of this table, an attempt is made to analyse and compare the different viewpoints forming the basis of the various Codes.

2.2. List of Codes.

C.1.	ÖNORM B 4600 : Stahlbau. Ausführung der Stahltragwerke, Teil 7 (1972)	Austria
C.2.	NBN-51-001 : Charpentes en acier (1977)	Belgium
C.3.	DASt-Richtlinie 012 (Deutscher Ausschuss für Stahlbau) (1978)	West Germany
C.4.	CNR - Bulletino ufficiale (Norme tecniche)-Nervature di irrigidimento delle anime di travi a parente piena - Anno VIII - (Nov. 1974)	Italy
C.5.	Norme SIA 161 - Constructions métalliques (1979)	Switzerland
C.6.	MERRISON Rules : part IV: Materials and Workmanship (1973)	United Kingdom
C.7.	Stålbyggnadsnorm 70 St BK-N1	Sweden
C.8.	European Recommendations for Steel Constructions (E.C.C.S.) (1978)	
C.9.	NS 3472 : Prosjektering av Stålkonstruksjoner Norges Standardiseringsforbund	Norway

C.10.Standard Specification for Highway Bridges, 12th ed.1977(AASHTO) U.S.A.

C.11.Montage des ponts métalliques; SETRA ; Bulletin technique N° 8 (1973) France

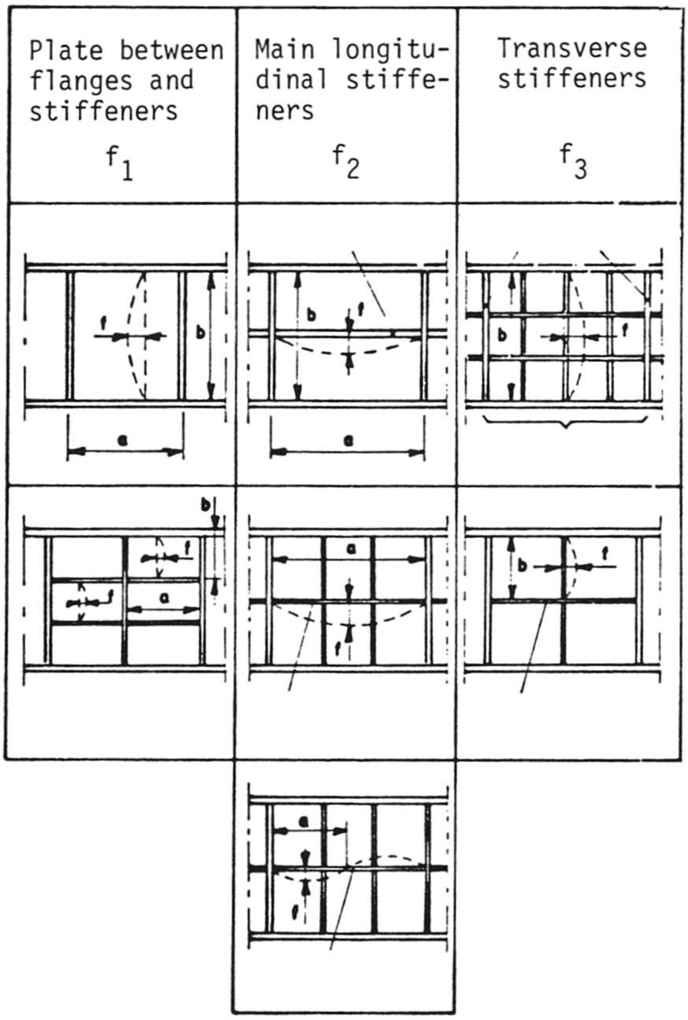
C.12.BS 5400 Steel, Concrete and Composite Bridges - Draft (1979) United-Kingdom

C.13.Draft of Design Specifications for Steel Box Girders. June 1979 Final Report, Ed. Dept. of Transportation, Federal Highway Administration, Washington, D.C. U.S.A.

2.3. Tolerances for stiffeners and plate panels given by different countries.

Table 1 contains out-of-plane deviations of plates and stiffeners which are specificied in several standards.(f_1 for a plate panel, f_2 for a longitudinal stiffener and f_3 for a cross-girder or a cross-frame, see figure 1).
The out-of-straightness imperfections parallel to the plate on the stiffener outstand are stated only in the Merrison Rules and so they are not given in this table.

When two values are given, the smaller of them is valid.



- Fig. 1 -

TABLE 1

N°	Code	Country	Permissible out-of-plane deviations f_1 mm	f_2 mm	f_3 mm	Note
C.1.	ÖNORM B 4600	Austria	a/250 or b/250	a/500 (x) max: 8 mm	b/500	a = length of stiffener or length of the half wave of stiffener buckling mode
C.2.	NBN B51-001	Belgium	a/250 or b/250 max: 4 mm	a/500 or b/500 max: 8 mm	b/500	
C.3.	DASt 012	West Germany	a/250 or b/250	a/400	a/400 or b/400	
C.4.	C N R	Italy	a/400 or b/400	a/500	b/500	stated for web panels
C.5.	SIA-161(1979)	Switzerland	a/250 or b/250	a/400	a/400 or b/500	for unstiffened webs of plate girders, the maximum out-of-plane deflections f are prescribed with reference to a gauge length of 2 m f = 5 mm for railway bridges ; f = 8 mm for highway bridges.
C.6.	MERRISON RULES N.B.: for more details, see the original reference[22]	United Kingdom	for t < 25 mm (x) $G(1+b/5000)/30t$ for t > 25 mm $(G/750)(1+\frac{b}{5000})$(xx) in the case of butt welds $\frac{t}{6}$ + 2.0 mm but not more than t/3 (t=plate thickness)	-a/1200 or +a/900 but not less than 2 mm	$\frac{l_1+l_2}{1000}$	(x) G is the gauge length = 2 b for a > 3b = a for a < 3b (xx) - max 1 mm in flange and diaphragm panels and in unrestrained web panels in compression - max 3 mm in other web panels
C.7.	St. BK-N1	Sweden	b/150 (x)	a/600 or b/600 (xx)		(x) valid for the web of a beam subjected to a bending moment (xx) valid for the bar subjected to compression.
C.8.	European Re-comm.for Steel Constructions	ECCS	a/500 or b/500	a/500 or b/500	b/500	
C.9.	NS 3472	Norway	b/133 (x)	a/1000		(x) valid for the web plate.
C.10.	AASHTO	U.S.A.	$\frac{0,159\ a}{144\ \sqrt{t}}$ (m) (x) or 4,8 mm	a/480	b/240	(x) These tolerances are valid for orthotropic deck bridges only.
C.11.	Montage des Ponts Métal-liques SETRA Bull.techn.8	France	$\pm\frac{1}{3}$(t+40mm) $\pm\frac{6}{3000}$ or $\pm\frac{1,5t}{10000}$ (x)		b/100	(x) for orthotropic bridge deck

N°	Code	Country	Permissible out of plane deviations f_1 mm	f_2 mm	f_3 mm	Note
C.12.	BS 5400 Draft 1979	United Kingdom	$\frac{2b}{165}\sqrt{\frac{\sigma_r}{355}}$ (a > 2b) $\frac{a}{165}\sqrt{\frac{\sigma_r}{355}}$ (a < 2b) but not less than 3 mm (x)	$\frac{a}{750}$ but not less than 2mm(xx)	$\frac{b}{750}$ (xxx) but not less than 2 mm	(x) applicable when $\frac{b}{t} > 25\sqrt{\frac{355}{\sigma_r}}$ (xx) for box girders and orthotropic decks (xxx) b = average spacing of cross-girders (5 % of the components in stiffened steelwork and 10 % of components in transversally stiffened steelwork shall be measured).
C.13.	Draft of Design Specifica-tions for Steel Box Girders	U.S.A.	1)bottom flanges of box girders a/200 or b/200 2)orthotropic decks $a/906\sqrt{t}$ or $b/906\sqrt{t}$ (in m) or 4,8 mm (x) 3)webs a/61 to a/130 or b/61 to b/130	a/500 (xx)	b/250	(x) These provisions for orthotropic decks [31] have been, for the time being, taken over from the AASHTO provisions now in force. However, it is suggested in the commentary of [31] that in the future these provisions be replaced by simpler rules, such as given for "bottom flanges", except that provisions for top decks should be much lenient, since design of decks is governed by local flexural stresses and axial compressive strength is of secondary importance (xx) measurement of f_2 shall include the effect of vertical curvature of the flange, if any.

Note : The attention of the reader should be brought to the fact that the tolerance measurements f_1 in the British Specifications C.6 and C.12 are made differently from all other specifications (see also the first paragraph of page 10).

2.4. Commentary.

Table 1 shows that the fabrication tolerances are very different in the various examined codes and that they depend on the experience and traditions of the fabricators.

The rules drafted immediately after the accidents (MERRISON Rules) are in main cases unacceptable for the fabricators and result in too high total cost of the completed steel work for carrying a given load. Such exacting tolerances would result in uneconomical construction as the unit cost of fabricated steel will increase substantially. On the contrary, in the new draft of the British code for tolerances, the tolerances are at levels that can be easily achieved.

Many codes, either don't differ for various types of bridges ([C.1] [C.3],[3]), or prescribe maximum imperfections for the webs of the plate girders only [C.4].

U.S.A. draft permissions for box girders [C.13] wish to cover all steel plate box girder bridges. For box girders, prescriptions differ for bottom flanges and orthogonal decks. They are also different from AASHTO Specifications, which prescribe the maximum out-of-flatness for "orthotropic deck superstructures" only.

Plates out-of flatness tolerances.

Most of the codes prescribe the maximal deflection relating only to the width of the panels. This ratio in most cases is b/250.

Such Codes as [C.13] and [C.1] are somewhat more liberal with value b/200. According to WOLCHUK's opinion [31], this value is appropriate as it satisfies the assumptions made in determination of strength.

AASHTO Specifications [C.10] for out-of-flatness are liberal, especially for subpanels of webs or flanges with close spacing of stiffeners, where the minimum value of 3/16 in governs. For example, if stiffener spacing, b, is 18 inches (460 mm), the permissible tolerance is b/96.

MERRISON Rules [C.6] and some Codes : [C.10] [C.11] take into account the influence of the thickness of the plate too. In the MERRISON Rules, the required tolerance is then related to the slenderness of the plate. It seems to be a little unpractical and too severe from the point of view of strength. For the small slendernesses ($b/t < 30$), where the effect of the initial bow isn't large, the MERRISON formula yields limit deflections relatively smaller than for thickness ratios higher than 30, where the effect of initial imperfection is large(up to $b/t = 100$), as table 2 shows :

Table 2

b = 500 mm

t(mm)	b/t	f/b
10	50	1/136
20	25	1/273

It should be acceptable in view of the considerable simplicity to present the tolerance as a direct function of the panels width. Note also that definitions and methods of measurement of plate out-of-flatness are different in various Codes.

The MERRISON Rules [C.6] and the new draft of the British code [C.12] propose an original method for measuring out-of-flatness and for checking the governing "ripple" component. The comparison with the other Codes is not possible because of differences in out-of-flatness definition and method of measurement. In german, belgian and ECCS Specifications, the out-of-flatness is defined as the maximum offset from the line perpendicular to the longer edge of the panel.

Stiffeners out-of-straightness tolerances.

The permitted value of the deflection of a longitudinal stiffener is generally related to the span a. The span - deviation ratio varies in the large bracket 400 to 1200 with a most frequent value of 500.

As is told in paper [31] of WOLCHUK, when a shorter gage length, ($G < a$) is used in establishing the stiffener tolerances as is permitted in [C.10], the actual out-of-straightness of the stiffener at mid-length will be greater than a/480, which may be unsafe for compression members. The writers of [C.13][31] eliminated the possibility of such a case by stating in proposed provision that the gage length must always be "a" and not shorter.

3. PROBLEMS RELATED TO THE GEOMETRIC MEASUREMENTS ON PLATED STRUCTURES AND THEIR EXPLOITATION.

3.1. How to make the measurements.

There are several methods to measure the geometrical imperfections of the different components or subcomponents of plate and box girders. The simplest method consists in measuring the deviations between the considered element and a reference line materialized by a taut nylon thread or a good ruler. It is a cheap and quick technique, but there is a lack of accuracy in the case of large components.

A second manner is to use a mechanical system consisting of a bar carrying two fixed probes and a central dial gauge (see [C.6]). For measuring ripples in plate panels, the gauge bar must be situated parallel to the longitudinal stiffener.

A third manner is the materialization of a reference plane (plane of sight of a theodolite) and the determination of the distances of points of the chosen element (web of a beam for example) from this plane. This procedure is very accurate, but requires a highly qualified staff. Moreover, the measurements can only be done when the element has a well determined position.

With a photogrammetric procedure, it is possible to obtain the whole map of the geometric deformations, while, with the other methods, only isolated values are obtained. But this method of measurement is very expensive and is not always easy to use on account of the necessary photographic distance. The accuracy is about one millimeter, which, in many cases, is insufficient.

A better method consists in recording electronically, on a X-Y plotter, the initial distortions of transverse or longitudinal sections of the stiffened plates and the increase of those distortions under load. Actually, this method is especially useful for measurements on laboratory models and it has been used successfully by Dr. R. MAQUOI and the chairman in their large size models on stiffened box girders [7] and is recommended by SPENCER [8], [9].

The measured geometrical imperfections must be referred to a base gauge length. These gauge lengths are generally a, b (or 2b).

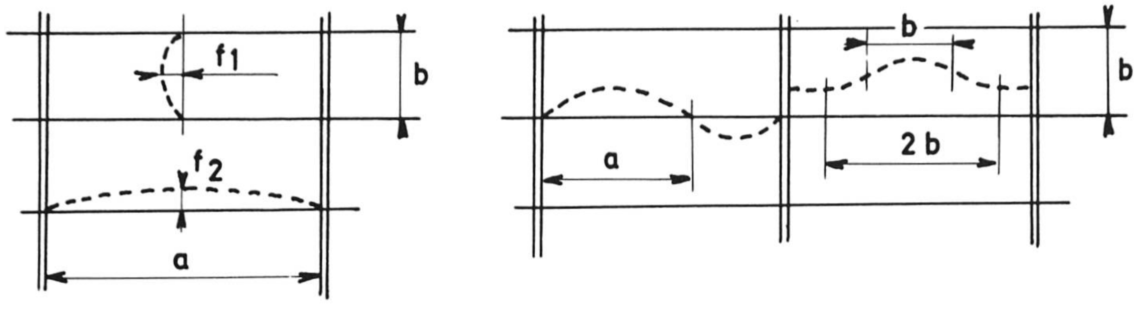


Figure 2

The measured imperfections of longitudinal plate stiffeners have generally one of the five following shapes and, for this reason, special attention must be given to the measurement and its interpretation.

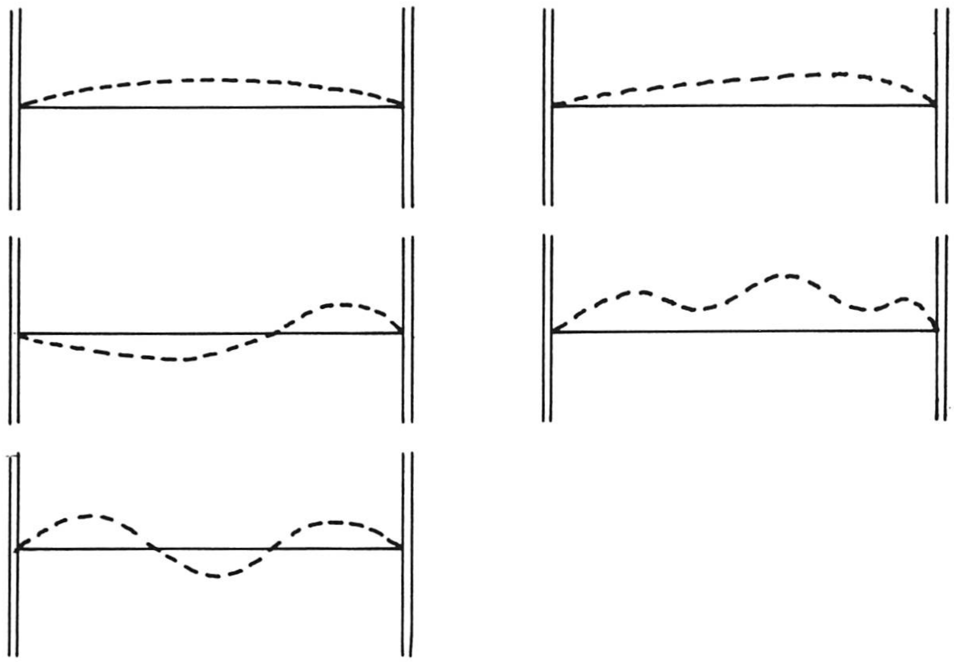


Figure 3

As we shall see hereafter in section 3.2., a recording of the deflections under increasing load is almost compulsory if one wants to determine the "equivalent" imperfection.

After measurement, it is interesting to determine from the obtained data the type of distribution, its central tendency and scatter. The probability density distribution of the data is generally approximately a normal LAPLACE-GAUSS one. However, from the analysis of the measurements, it can be seen that this is not always the case and that the resulting distribution may be a log-normal or a WEIBULL distribution.

3.2. How to exploit the measurements ?

3.2.1. Unloaded panels.

The imperfections to be extracted from the measurements must give a realistic measure of the magnitude of the imperfection. As, in steel plated structures, the degree of postcriticality

$$n = \frac{\sigma_{collapse}}{\sigma_{cr}} \tag{1}$$

(where σ_{cr} is the critical buckling stress given by the linear buckling theory) does not usually exceed 1.5, it can be admitted (for a deeper study of this point, see e.g. the book by WOLMIR [10]) that the deflection mode is nearly affine to the first eigenmode of the linear buckling theory (x). That is the reason why the measurements should be used to establish the magnitude of the first buckling mode. For this purpose, it is clear that - in the simple case of a stiffener - it will not suffice to measure the deflection in the middle, M. Indeed, this deflection might, per chance, be zero, which would mean that the imperfection is nearly orthogonal to the first buckling mode (shown dashed on fig. 4).

- Fig. 4 -

An interesting paper discussing this problem was very recently published by SPENCER [8][9]. He analyzed the postbuckling behaviour of a rectangular plate panel subject to uniaxial uniform compression. Using the perturbation method in integrating the von KARMAN-MARGUERRE non linear equations of membrane-plates, one obtains easily the following non linear relation

$$(W/t)(W/t + 2\,W_o/t) = A^2\,[P/P_c - W/(W + W_o)] \tag{2}$$

between t, the thickness and W_o, the amplitude of the initial deflection of the panel

$$w_o\,(x,y) = W_o \cos\frac{\pi x}{a}\ \cos\frac{\pi y}{b}\,, \tag{3}$$

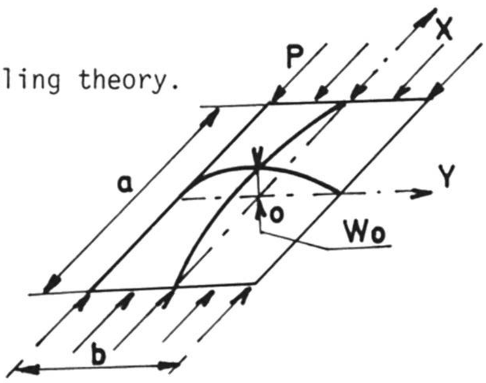
W, the amplitude of the additional deflection

P_c, the critical load given by the linear buckling theory.

$P, (> P_c)$ the postbuckling load

A, a parameter

Now, from sets of experimental data (P,W), it is possible to determine the three constants P_c, A and W_o which give the best fit with the experimental curve by using the OPLS non linear least squares program [11].



- Fig. 5 -

(x) We don't cover here the case of light gage steel or of especially slender beams like the Swedish HSI-girders, where the degree of postcriticality can reach 10 or 3 respectively.

The advantage of this procedure is that, because it is based on curve fitting, it finds the effective imperfection W_o which takes into account not only the initial geometric imperfections but also the load eccentricity etc... The method presented above for an isotropic plate can be generalized for orthotropic plates reinforced by one or two orthogonal families of eccentric stiffeners, provided these stiffeners are supposed to be "smeared up".

As a matter of fact, starting from the equation (7.10.) (page 106) established in 1971 by Dr. MAQUOI and the chairman in their non linear theory of the behaviour of compressed stiffened flanges [12], it is easily possible to show :

a) that the equation reduces to SPENCER's if no stiffeners are present (isotropic plate) ;
b) that, in the case of stiffened plate, the error of SPENCER's approach ought to be very small.

However, if SPENCER is quite right in preferring to deal with effective imperfections obtained from the actual response of the structure under load, his preference is, unfortunately, largely academic. Indeed, it is - financially - almost impraticable to make load deflection measurements under continuously increasing load in actual bridges and, even if this were possible, one could not record the effects of the dead weight. On the other hand, P-δ measurements obtained on small models such as those described in SPENCER's paper [9] are hardly valid, because it is well known that similitude applies very badly to structural instability tests.

For plate panels, the method of measurement can basically influence the results. When the total distortion f_{max} is measured relatively to the longitudinal stiffeners (gauge bar is situated perpendicularly to the longitudinal stiffeners), the results will be different from the deformation f' (=2Δf)measured in the longitudinal direction on a basis 2b (fig. 6.a.). Indeed, the diagram of figure 7.a.(giving the buckling coefficient k as function of the side ratio α) shows that, for large values of $\alpha \equiv a/b$, like 3 for instance, the third buckling mode (m = 3) gives the least critical stress

$$\sigma_{cr} = k\,\sigma_E = k\,\frac{\pi^2 D}{b^2 t}$$

and that the other modes (m = 1, 2, 4, etc...) give much larger values.

The Δf ripple amplitude may be superimposed on a transverse curvature, whose shape is uniform along the full length of panel. This "hungry horse" shape is induced by shrinkage across the plate-stiffener fillet welds. This curvature is not a weakening effect and tests by LITTLE and DWIGHT [23] confirmed this statement. BRADFIELD [24] proposes, on the basis of measurements, the effec ive value of $\Delta f = \frac{b}{1000}$ for check of the plate; this value corresponds to the method of calculation proposed by DWIGHT and LITTLE. CARLSEN [13] recommends to measure the mode of distortion shape of the panel in as many points as to be able to express the deformation shape by a double trigonometrical series

$$w_o(x,y) = \sum_{i=1}^{m} \sum_{i=1}^{n} A_{ij} \sin\frac{i\pi x}{a} \sin\frac{j\pi x}{b}$$

and find the FOURIER component coinciding with the buckling mode component. However, to get a reliable estimate of the amplitude of the buckling mode component, the number of measurement points should be more than the number of half waves of the component. This procedure may be recommended for the statistical establishment of a relation between the buckling mode component of the imperfection and a characteristic parameter f_{max} which is easy to measure.

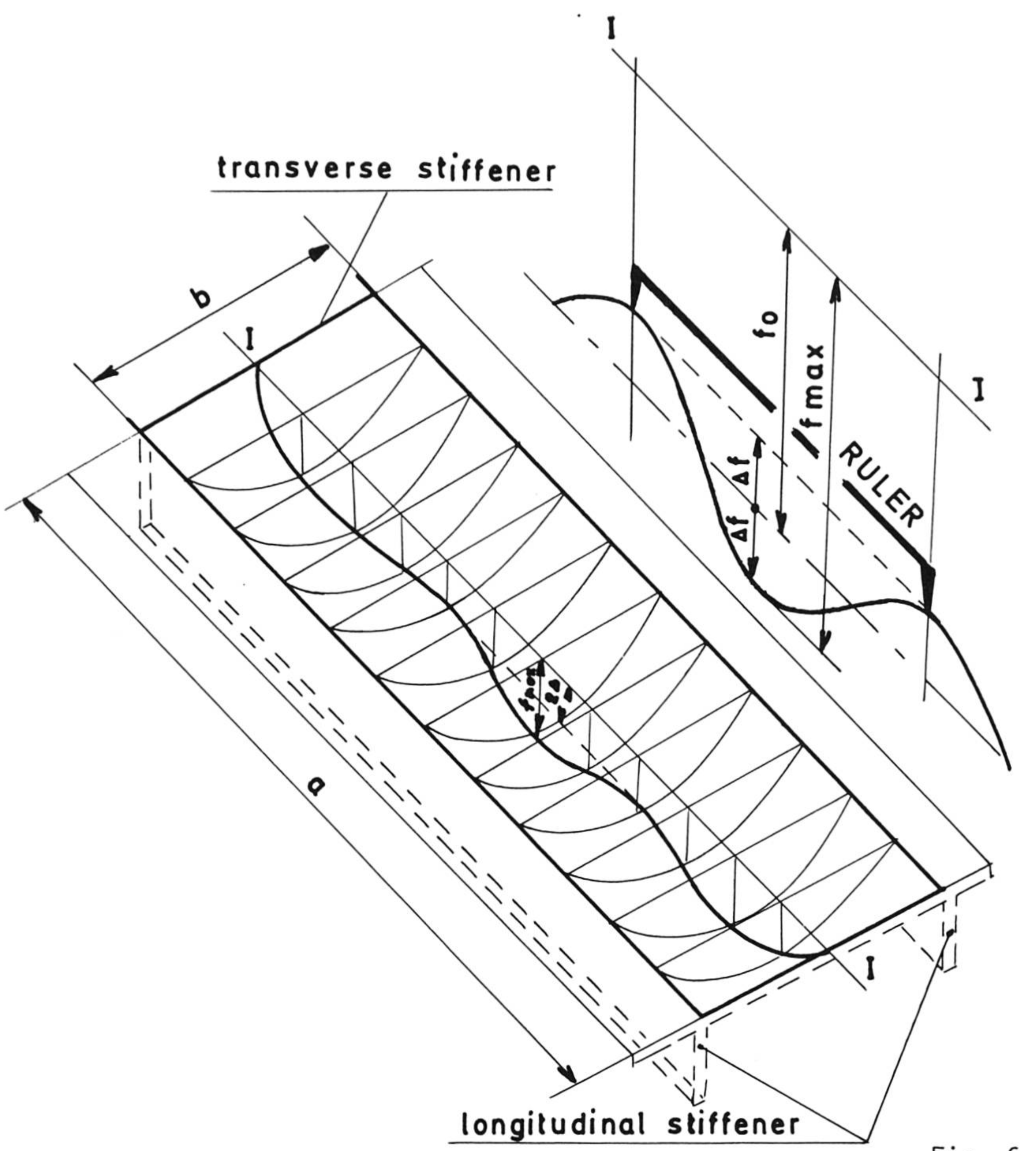


Fig. 6.a.

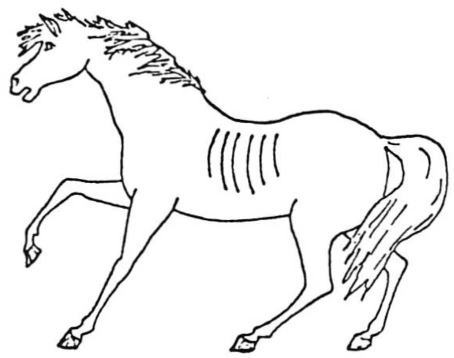


Fig. 6.b.

Cross section of plating showing 'hungry horse' shape

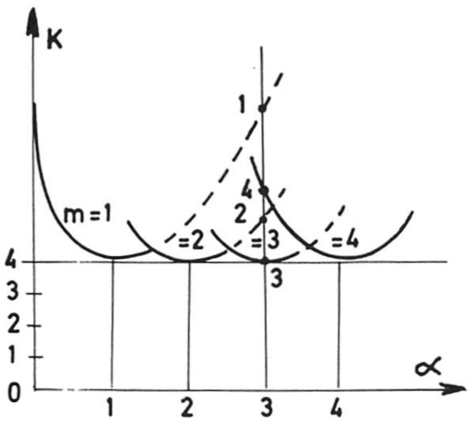


Fig. 7 a

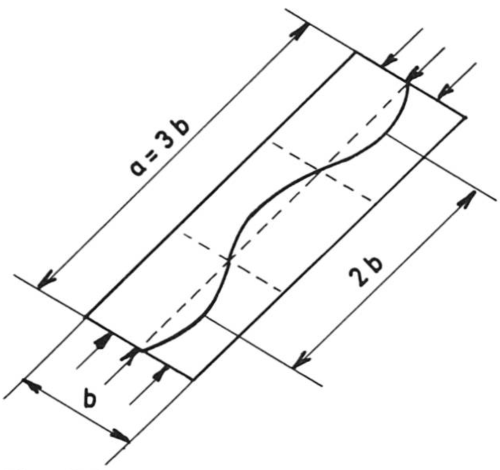


Fig. 7 b

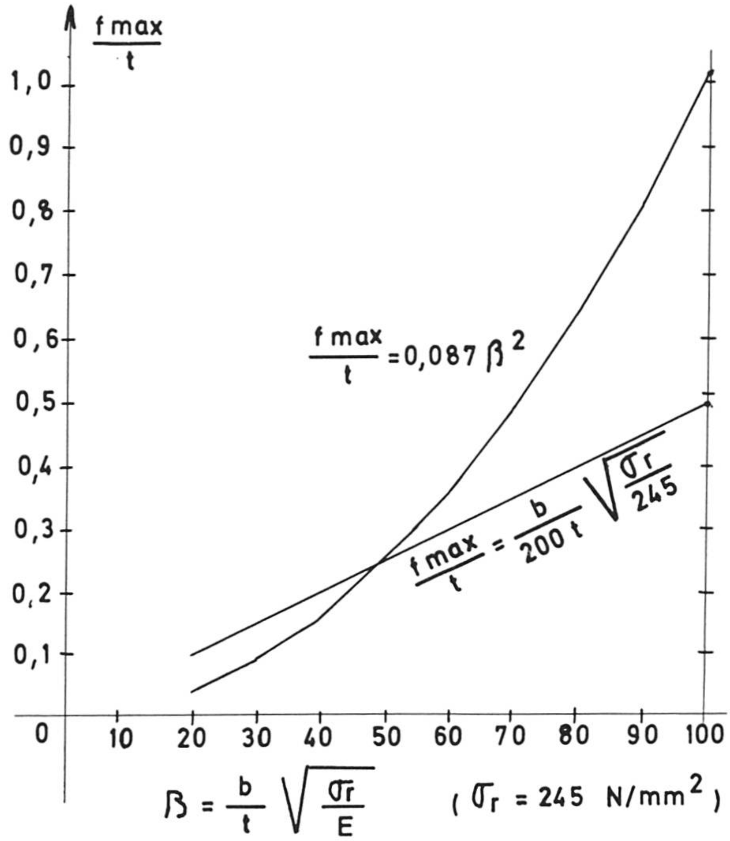


Fig. 8

On the basis of CARLSEN measurements [13], this relation is $\Delta f = \frac{1}{2} f_{max}$. The measured values of f_{max}/t are in good agreement with the value $\frac{b^2}{200}$ proposed by DOWLING [27]. FRIEZE [25] proposes $f_{max}/t = 0{,}087\ \beta^2$. As the comparison of following figure 8 shows, in the most sensitive region of thickness ratios b/t (b/t = 50), the proposals of DOWLING and CHATTERJEE give the same results.

Therefore, the practical results of above discussion should be as follows :

1) utilize large size model tests or full size tests (such as those of Imperial College) to record carefully (P,δ) diagrams under increasing load.

2) If conventional "isolated" measurements are performed, try to determine the amplitude of the first buckling mode.

From the 1979 discussion of CARLSEN's paper [13], to which most of the british specialists took part, it appears that opinions about the manner to measure the plate imperfection are highly differing. HORNE and CHATTERJEE at least recommend to measure imperfections 2Δf of *long* plates in the *longitudinal direction* over a gauge length of 2b, not b. The reason for this gauge length is shown clearly in the upper part of fig. 6 a : If buckles are approximately square and alternatively up and down, a ruler of length 2b will measure an imperfection of 2Δf, equal to twice the "ripple" imperfection.

3.2.2. Loaded panels.

In Belgium, the measurements of the imperfections were made in the fabrication plant, while, in Czekoslovakia, United Kingdom and Western Germany, they were performed on the erected bridges subjected to their dead weight. For long span bridges, it is easy to show that these two types of measurements cannot be directly compared. Indeed, taking as a basis the linear theory of imperfect structures, we know from classical treatises that

$$W_{(\text{dead load})} = \frac{W_o}{1 - \frac{\sigma}{\sigma_{cr}}}$$

where W_o is the initial deflection in the (fabrication) unloaded state ;

$W_{d.1}$ the total deflection under dead load ;

σ the stresses in the panel due to dead load ;

σ_{cr} the critical stress of this panel.

Therefore,

$$W_o = W_{d\ 1}\ (\text{measured})\ (1 - \frac{\sigma}{\sigma_{cr}})$$

It is obviously impossible to compute the minoration coefficient $(1 - \frac{\sigma}{\sigma_{cr}})$ for all panels of the many bridges inspected because we have not the corresponding data. However, the Bureau of the Task Group performed some computations for a box girder.

The cross section of the box girder is represented at fig. 9.

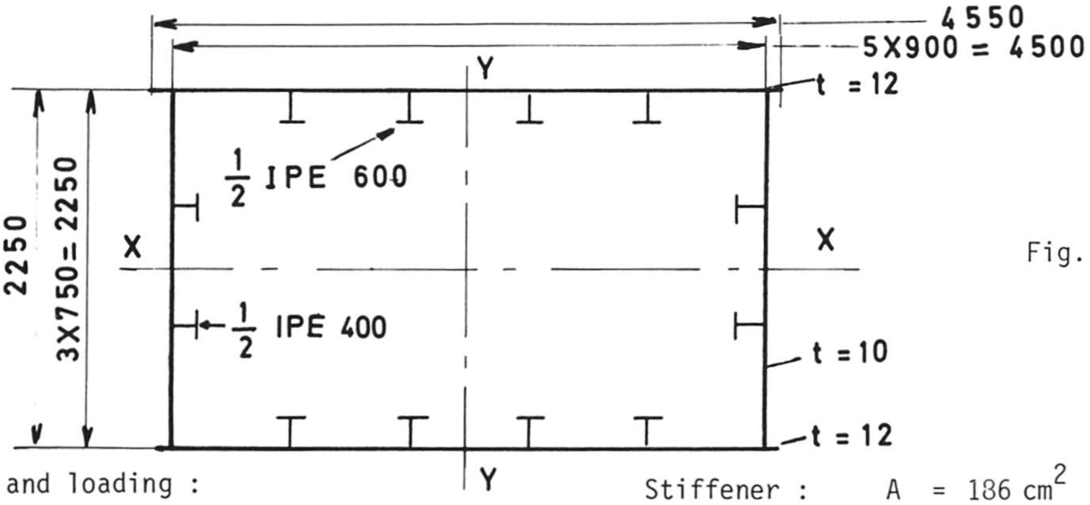


Fig. 9

Span and loading :

1) Box girder : l = 54 m

2) Stiffener a = 6,75 m

Dead weight g = 20 kN/m (σ_g = 39,2 N/mm^2)

Cross sectional characteristics

$A = 2332{,}4\ cm^2$

Box girder : $I_x = 2092\ cm^2\ m^2$

$W_x = 1860\ cm^2\ m$

Stiffener : $A = 186\ cm^2$

$I = 30710\ cm^4$

$i_x = 12{,}85\ cm$

$\lambda = \frac{a}{i_x} = 52{,}52$

Plate panel : $a = 675\ cm$

$b = 90\ cm$

$t = 12\ mm$

For a longitudinal stiffener, it was found that, in the middle section of the bridge,

$$W_o = 0,95\ W_{measured}.$$

For plates, we can only use this formula in the case of the "ripple component" of the imperfection. Indeed, fig. 6 and 7.b. show that the most amplified buckling mode will correspond to the ripple in question. For the bridge in question, we find :

$$W_o = 0,71\ W_m.$$

In conclusion, a small correction should be applied on the measurements under dead load, but this correction is likely to be less than 5 per cent.

3.3. Results of measurements made on steel road bridges.

A large number of measurements of geometrical imperfections have been reported in the literature. The majority of these measurements were made on panels prepared for experiments and they are not representative of the workshop production (scale, welding procedure, etc...).

In 1970, global imperfections of the bottom flange of 2 large box girder bridges have been measured in Belgium [14] and it was shown that the initial camber to be considered for the study of the postbuckling behaviour of the compressed panels could be equalized to the thickness of the sheet.
Nevertheless, up to 1975, one did not know enough about imperfections which occur as a result of the fabrication process of steel structures. To improve this lack of information, measurements of imperfections on panels, subpanels and stiffeners of steel road bridges were made in several countries as, for example, Belgium, Czekoslovakia, West Germany and United Kingdom.
The term of imperfection includes geometrical tolerances and residual stresses.

The road bridges suitable for the survey in the different countries were selected according the following criteria :

- bridge system ;
- different steel fabricators and erectors ;
- type of stiffening ;
- dimensions of plates and panels.

Measurements were made of the deflections of plate panels, stiffeners and stiffened plate panels; the deflection being that a mid-point of the gauge length. The following tables 3.1. and 3.2. give the condensed results of some of those measurements, namely the mean value and the standard deviation of the maximum deformation of the subpanel plates and the longitudinal stiffeners. All these values have been normalized to the same gauge lengths "a" or "b", i.e. the dimensions of a subpanel (fig. 10).

Figure 11 gives the distribution (in %) of the measured imperfections of the longitudinal stiffeners in 3 of the 4 countries considered.

The belgian survey examined imperfections during fabrication while, in Germany and United Kingdom, the bridges were in service at the time of the measurements with full dead load (and normal traffic loads in U.K.). This may be a partial explanation of some good performances of the belgian results (95 % fractile). Another reason is that, since the publication of the draft of Code NBN 51-001 "Charpentes Métalliques", a special attention was given and a severe control enforced for the respect of these requirements. It should be stated that, in Belgium, the same measurements will be made on the same bridges in service and

Table 3.1. Measured deformations at the centre of sub-panels (f_1)

COUNTRY	Number and type of bridge	Position of the subpanel	number of measurements	thickness of the plate (mm)	f_1/b	$s(f_1/b)$	95% fractile	Tolerance of code
BELGIUM [15]	1 box girder bridge (measurements before erection)	web	166	12 - 32 mm	1/518	1/1483	1/315	1/250
		bottom flange	83		1/479	1/1079	1.265	
		Total	249		1/505	1/1296	1/308	
	1 plate girder composite bridge (idem)	web	1462	12 - 50 mm	1/724	1/713	1/241	
	TOTAL OF THE 2 BRIDGES		1711		1/680	1/764	1/249	
CZECHOSLOVAKIA [16]	4 box girder bridges	bottom flange	96	16 mm	1/998	1/680	1/381	1/250
			64	25 mm	1/491	1/325	1/191	
			37	30 mm	1/397	1/236	1/151	
			71	25 mm	1/705	1/551	1/259	
			39	30 mm	1/600	1/458	1/226	
			36	25 mm	1/213	1/438	1/147	
			23	30 mm	1/622	1/534	1/192	
			37	14 mm	1/253	1/465	1/127	
			53	20 mm	1/479	1/449	1/214	
			312	12 mm	1/305	1/284	1/113	
			299	20 mm	1/333	1/246	1/115	
			280	32 mm	1/237	1/295	1/146	
			270	14 mm	1/289	1/216	1/109	
			1617		1/456	1/292	-	
UNITED KINGDOM [17]	7 box girder bridges	web and bottom flanges	5884	-	0,000295(x)	0,001588	1/275	1/165 for higher yield steel 1/200 for mild steel
WEST GERMANY [18]	2 box girder bridges	web	2022	10 - 18 mm	1/207	1/448	1/106	1/250
		bottom flanges	469		1/384	1/384	1/127	
		Total	2491		1/226	1/434	1/110	

(x) the mean value given here takes account of the sign of the measured imperfections.

Table 3.2. Measured deformations of longitudinal stiffeners (f_2)

COUNTRY	Number and type of bridge	Position of the stiffener	Number of measurements	thickness of the plate	f_2/a	$s(f_2/a)$	95% fractile	Code'
BELGIUM [15]	1 box girder bridge(measurements before erection)	web	166	12 - 32 mm	1/1710	1/3198	1/895	1/500
		bottom flange	83		1/1509	1/3168	1/817	
		Total	249		1/1642	1/3162	1/887	
	1 plate girder composite bridge (idem)	web	1239	12 - 50 mm	1/2294	1/2343	1/667	
	TOTAL OF THE 2 BRIDGES		1488		1/2150	1/2439	1/671	
CZECHOSLOVAKIA [16]	4 box girder bridges	bottom flange	42	16 mm	1/835	1/681	1/305	1/500
			216	25 - 30 mm	1/844	1/1137	1/400	
			60	14 - 20 mm	1/497	1/719	1/240	
			130	12 - 32 mm	1/333	1/312	1/142	
		Total	448		1/548	1/596	-	
UNITED KINGDOM [17]]	7 box girder bridges	web and bottom flange	1614		0,000024(x)	0,000681	1/740	(1/750)
WEST GERMANY [18]	2 box girder bridges	web	202	10 - 18 mm	1/640	1/1170	1/306	1/500
		bottom flange	175		1/708	1/1371	1/348	
		Total	377		1/667	1/1250	1/324	

(x) the mean value given here takes account of the sign of the measured imperfections.

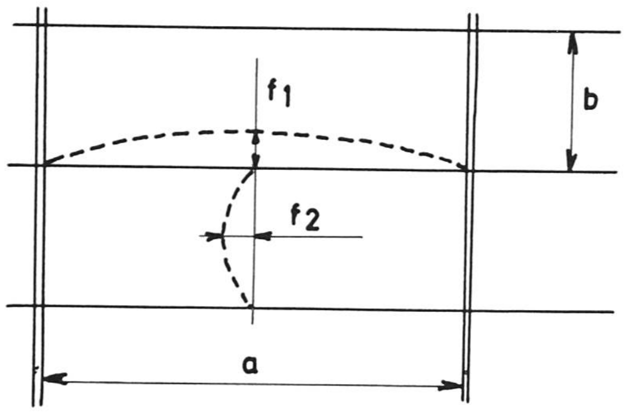


Fig. 10

an interesting comparison will then be possible between the two types of measurements.

The comparison of the german measurements with the fabrication tolerances of DIN 1079 shows that, in the case of longitudinal stiffeners, 21 % of all measurements have not met the requirement $f_2 < \frac{a}{500}$ and, in the case of subpanel plates, 50 % of all measurements of web elements and 25 % of all measurements of lower flange elements have not met the requirements $f_1 < \frac{b}{250}$.

In the United Kingdom, the Department of Transport obtained approximately 14000 measurements on the plates of seven box girder bridges. The imperfections were measured in three different manners, as indicated by Table 4 [13 bis].

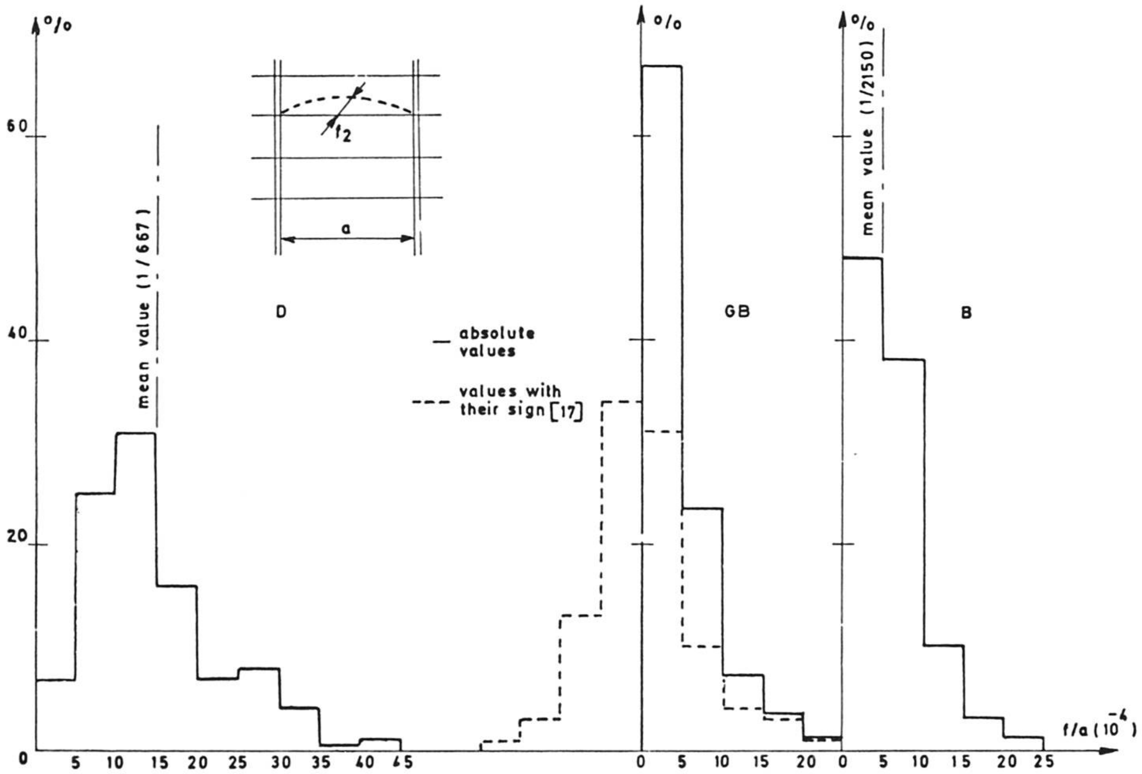


Fig. 11

Table 4

Measurement method	Number of measurements	95 % fractile value
A Over b longitudinally	5884	b/320
B Over 2b longitudinally	2186	2b/425
C Over b transversely	6328	b/275

From the measurements, the Department of Transport proposes a linear relationship between f_1 and b : $f_1 = \frac{b}{200} \sqrt{\frac{\sigma_r}{245}}$. Since plates of higher grades of steel are likely to be thinner for resisting a given load, this approach should be welcome to fabricators.
For the longitudinal stiffeners, it was shown that 95 % of the stiffeners satisfy a tolerance of span/740. From these results, the Department of Transport proposes for the new British Code a value of span/750 instead of the proposed tolerance of span/900, which seems to be too strict for the fabrication practice.

In Czechoslovakia, it was shown that, for the longitudinal stiffeners, according to the bridge where the measurements were made, there were 11,9 - 5,1 - 21,7 or 76,9 % of the stiffeners with a deformation larger than the allowable value of 1/500, while, for the bottom flange plates, there were from 2.8 to 45.9 % panels out of the tolerance of 1/250.
The statistical analysis of the czechoslovakian measurements was carried out on a computer for several distributions : normal distribution, log-normal distribution, distribution of minimum of maximum values. As the boundaries for each distribution curves differ a little from each other, their extreme and mean values (calculated for 98 %, 95 % and 90 % fractile) are given. In tables 3, the 95 % fractile is that of the mean value of the individual bridges.

The analysis of the german, belgian and czechoslovakian data does not permit to determine a significant influence of some parameters as the dimensions of the panels and the plate thickness. The results only indicate that the imperfections are mainly influenced by shop fabrication techniques (welding procedures) and erection tolerances.

From all these measurements on 14 steel road bridges, it seems that the imperfections are mainly influenced by the shop fabrication and erection techniques. In some cases, the imperfections are outside the allowable limits; but we should be careful before concluding, because we have not enough information regarding the way the measurements were analyzed (gauge length, type of deformations, etc.). There is a need for having more measurements and mainly for having a common technique for analyzing the results.

The value of the fractile which is to be recommended for the analysis is still disputable. BRADFIELD states in [24] that a low probability of failure has already been provided against variations in yield stress.
The variability of this swamps the effects of varying geometrical imperfections. To maintain the same probability when imperfections are included, the above very low probability of occurrence is not required. He proposes therefore the 0.001 b tolerance for calculating the strength of stiffened steel plating, in spite of the fact that 37 % of all measurements exceeded this value.

When large ripples occur, adjacent panels contain ripples such that these panels do not buckle sympathically. Plates containing large ripples have a more rounded maximum in their load-shortening curves and are better able to carry the load and allow for redistribution of load within the structure. Therefore, the 95 % fractile based on imperfections influenced by shop fabrication techniques seems to be a safe value taking also care of imperfections influenced by proper erection techniques.

4. EFFECTS OF IMPERFECTIONS ON WORKING STRENGTH AND ULTIMATE STRENGTH IN PLATED STEEL BRIDGES.

As stated clearly in Sections 1.2. and 1.3., what we need is :

- an assessment of the geometric and structural imperfections actually present in correctly fabricated steel bridges. Some information on that point has already been given in Section 3 ;
- an assessment of the effect of these imperfections on the ultimate strength of the structure. Here, the studies are only beginning.
- an assessment of the cost effect of requested tolerances, so as to obtain the optimal structure, in the sense defined in Section 1.3. This has not been made at all. It varies obviously very much with extratechnical factors, and mainly with the level of the salaries.

The Committee knows some papers ([19],[20],[25], [32],[33]), which analyze quantitatively the effect of imperfections on the ultimate strength of plate - and box-girders.

4.1.

The first paper [19], presented at the Liège Colloquium, due to Dr. DOWLING and his associates, contains several interesting conclusions, that may be summarized as follows :

<u>4.1.1. Sensitivity of (isotropic) plate panels.</u>

a.- for isotropic plates in uniaxial compression, the most sensitive b/t range extends both ways of the b/t value where the elastic critical buckling curve intersects the squash plateau for the grade of steel considered (that is b/t = 55 for mild steel) [19][24]; the same statement, incidentally, was made by MOXHAM several years ago [21].
According to the diagrams of fig. 12, taken from the paper [19], a doubling of the initial bow from b/400 to b/200 yields only a drop in strength of less than 10 %. A further doubling of the initial bow to b/100 produces an additional drop in strength of up to 12 %.

b.- for panels subjected primarily to shear, the effect of geometrical imperfections is very small for all slendernesses. A change of f_1/b from 1/200 to 1/50 produces a drop in strength of less than 9 per cent for the most sensitive case, namely b/t = 180.

These findings have been confirmed recently by Prof. F. FREY in his Ph. D. Thesis [26] (pp. 11.11. to 11.13.) in which he shows that very large initial imperfections do not affect the ultimate strength of unstiffened plate girders subjected primarily to shear.

c.- for plates loaded in compression and shear, an increase in initial bow (f_1) from b/200 to b/100 makes very little difference to the ultimate strength.

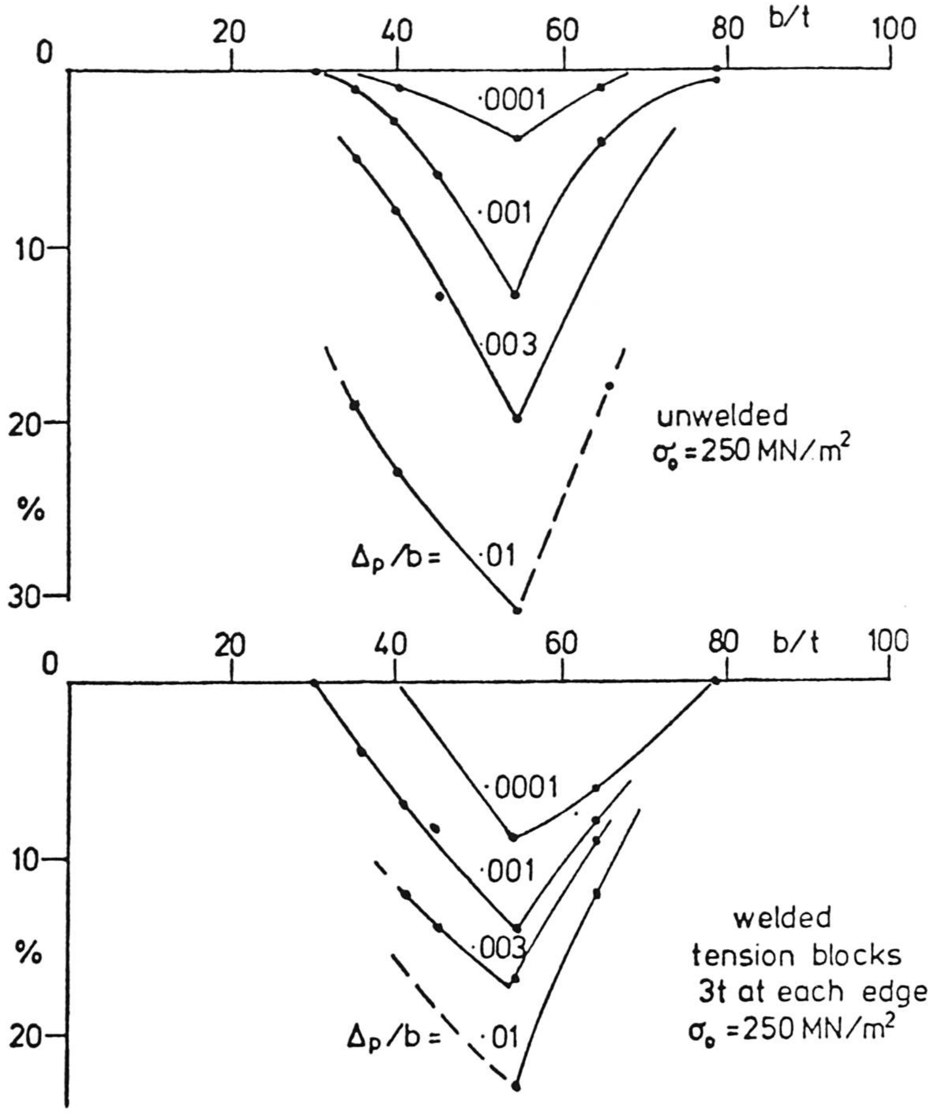


Fig. 12

d.- for plates loaded in bending and shear, a change of f_1/b from 1/200 to 1/50 yields a drop in shear carying capacity of less than 5 per cent, the worst case is again b/t = 180.

4.1.2. Sensitivity of stiffened compression panels.

For stiffened panels buckling in a practically cylindrical mode, that they treat like a series of parallel inelastic beam-columns, DOWLING and al show that, for the whole practical range of stiffeners, the effect of varying the plate initial imperfection on overall strength is negligible for initial bows even larger than b/200. An increase in stiffener bow f_2 from 1/1000 to 1/750 produces a maximum reduction in strength when the stiffeners fail by compression of the outstand, and is in all cases less than 10 per cent.

In their second paper [20] devoted to welded steel box girder flanges, which is partly based on measurements on ten quarter-scale box girder models, DOWLING and al first observe that "while the residual strains caused by welding the stiffeners to the plate may be predicted with a fair degree of confidence, it is also found that a considerable alteration to the distribution of strains may occur

while assembling the box flanges and webs, which is beyond accurate estimation". They show then, by computer calculations, that, "when the level of initial distorsion f is b/200, the steady decrease experienced with the initial increase in residual stresses rapidly reduces for residual stresses in excess of 10 per cent". A similar conclusion is obtained for stiffened flanges if the initial stiffener distortion f_2 does not exceed a/750.
This demonstrates that "an allowance for compressive residual plate strain of 10 per cent of the yield strain is sufficient to cover the weakening effects of welding residual strains on the strength of most practical welded box girder steel flanges, irrespective of the amount of welding residual strain actually present. The accurate preduction of such strains thus becomes unnecessary for design".
As conclusion of both paper [19][20], DOWLING and al recommend to use a tolerance of b/200 on the plate and a/750 on the stiffeners. They stress the fact that trained inspectors and experienced fabricators can spot such imperfections by eye and that therefore the elaborate checking procedures recommended by the MERRISON Rules [22] are not necessary.

Several other papers analyse theoretically the effect of imperfections on compressed panels.

4.2.

The doctoral thesis of B. ROUVE [32], gives (on page 129) a diagram which shows the loss of ultimate strength due to a series of geometrical imperfections W_o/b of various amplitudes. However, the analysis pertains only to purely elastic isotropic plates and residual stresses are disregarded.

4.3.

In a 1979 technical report [34], CARLSEN analyses stiffened plates in compression by the finite difference program STAGS and simulates their load-shortening curves by using stress-strain curves influenced by the presence of welding residual stresses. He reaches a series of conclusions, the most important of which are :

4.3.1. Structural behaviour.

A significant interaction occurs between adjacent stiffener spans for continuous stiffeners supported by transverse girders.

4.3.2. Geometrical parameters.

The primary geometrical strength parameters are stiffener and plate slenderness

$$\beta = \frac{b}{t}\sqrt{\frac{\sigma_r}{E}} \; ; \quad \overline{\lambda} = \frac{a}{\pi r}\sqrt{\frac{\sigma_r}{E}} \quad \text{where} \quad r = \sqrt{\frac{I}{A_s}}$$

4.3.3. Imperfection parameters.

The effect of initial stiffener deflections, expressed as fraction of the length f_2/a, increases with increasing stiffener slenderness. Thus, the reduction in strength for an increase of f_2 from 0.5 to 2.5 ‰ of "a" was for b/t = 55:

Table 5.

$\overline{\lambda}$	Strength reduction, %
0.3	3
1.0	16
1.5	26

The effect of plate distortion amplitude on the stiffened panel is moderate. The strength reduction for an increase of f_1 from 0.5 to 1.5 % of b was 5 - 10% with smallest reduction for high plate and stiffener slenderness.
The effect of welding stresses in the plate depends both on the plate and stiffener slenderness. For stocky plates, welding stresses beyond 10 - 15% of the yield stress have no further deteriorating effect.
This last conclusion is in line with that obtained above by DOWLING and associates.

4.4.

Using a special test rig of 1MN capacity at University of Cambridge, BRADFIELD [35] obtains the load-deflection relationships of 57 compressed plates of regular high-strength steel (Fe 510 - with σ_r^{min} = 345 N/mm^2), for various support conditions along the unloaded edges. Out-of-flatnesses are deliberately introduced and controlled. Residual stresses are also introduced by laying beads of weld along the unloaded edges.
In a comparison paper [36] , BRADFIELD and CHLADNY compare the experimental results of [35] with five large displacements elasto-plastic numerical methods, which all require large digital computers.
Fig. 13 compares the five theoretical predictions with the experimental results for f_{max}/b = 0,005. The tests show :

1) that it is possible, for an unwelded plate with $\beta^x < 0.7$, to develop the full squash load σ_r.A of the plate, even in the presence of above substantial initial out-of-flatness;
2) that, for normal welded compression panels of slenderness $\beta^x < 0{,}55$, the squash load may also be reached. For the domain of slenderness

$$0{,}6 < \beta^x < 1.1,$$

the ultimate strength depends on the degree of distortion. The experimental results are above the theoretical predictions by up to 10 % of σ_r.

4.5.

Finally, the behaviour of unstiffened compressed plates of side ratio $\alpha \equiv a/b < 1$ was carefully studied experimentally by FISHER and HARRE [33]. Fig. 14, taken from this reference, shows the proposal made by above authors. For $\alpha > 1$, the α = 1 curve is valid and is in good agreement with the findings of other authors.

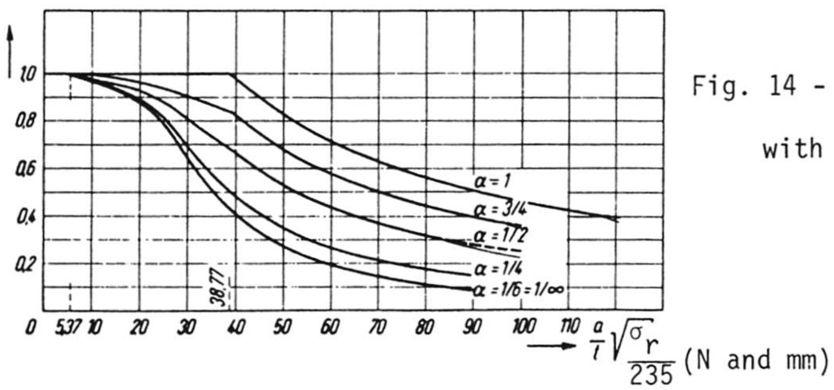


Fig. 14 - Load slenderness curves for unstiffened plates with sides ratio α < 1 [33].

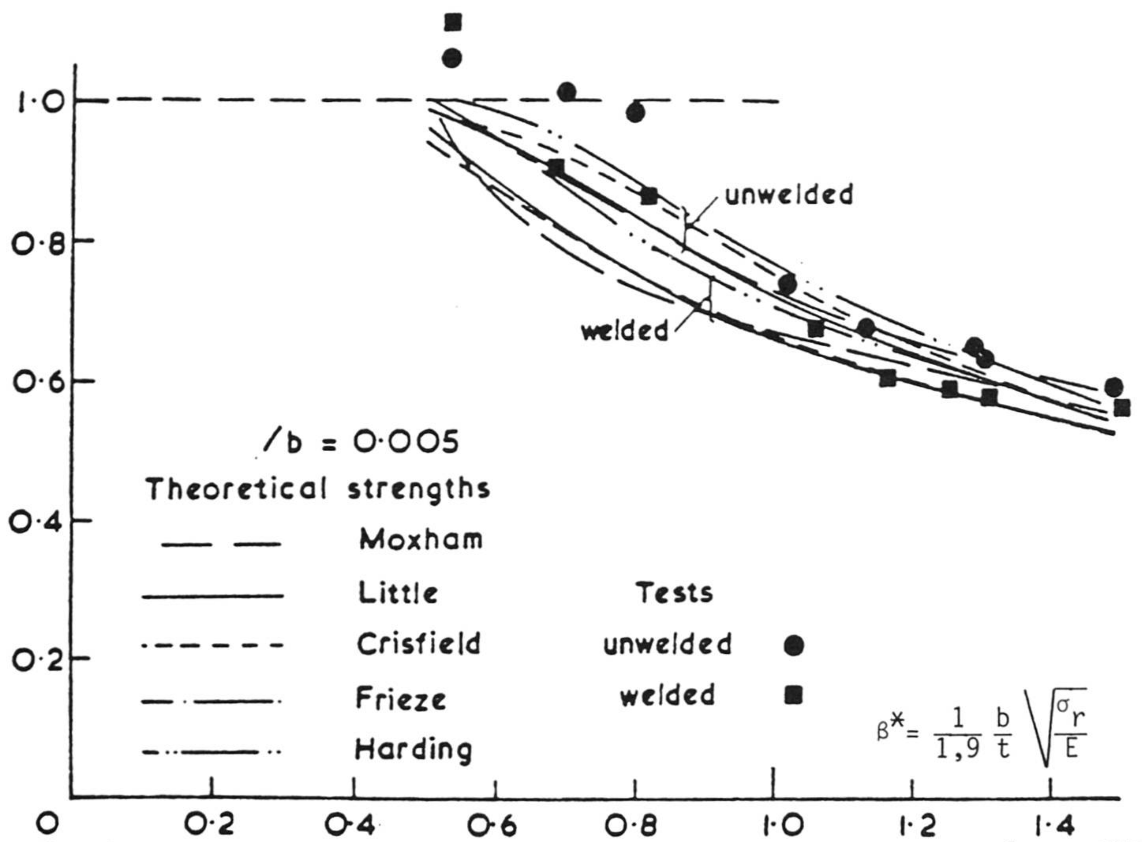


Fig. 13 - Comparison of experimental strengths with numerical analyses [35].

5. CONCLUSIONS AND RECOMMENDATIONS.

5.1.

In the design of orthogonally stiffened plates, rules must be adopted which include the imperfections in line with the procedure accepted by ECCS Recommendations for column design. At present, the design of the plates is generally based on the linear buckling theory, which assumes an "ideal" structure. The stiffened plates of the bridges, which generally have their stiffeners placed only on one side, cannot be fabricated without geometrical and structural imperfections due generally to welding. From an academic viewpoint, one should be opposed to the use of procedures eliminating the geometric imperfections after fabrication, because they introduce uncontrollable residual stresses and no evidence exists that the ultimate strength has been improved. However, fabricators will not uniformly agree with this requirement. The weakening effect of these imperfections on the one hand, which is the greatest for the middle b/t range (40 - 60) and the favorable membrane effect on the other hand for the greater b/t ratios in postcritic stadium are the main reasons why the linear buckling theory is not applicable.

Most of the ultimate design methods developed recently [12],[27],[18],[16],[30] include the effects of imperfections.
As the structural imperfections (residual stresses) are very difficult and costly to measure and, according to [27], in normal welded flanges rarely exceed 10 % of the yield stress, this value can be accepted in design rules. This value can simply be added to the applied compressive stress. Computations show that the ultimate strength of the welded plate will be the same as that of the unwelded plate [27].

5.2.

A set of realistic and easy to control tolerances should be established, which satisfy the requirements for the optimum structure i.e. the costs associated with the control and the fulfillment of the requirements of the tolerances must be in relation with the material cost and the safety of the construction.

The imperfections must be such that the workshops can respect them by working well without costly special equipment.

5.3.

A lot of measurements on full size bridges have been performed in four countries. Their results are collected and analyzed in present document.
As the results of these measurements (tables 3.1. and 3.2. in chap. 3) show, in these countries the requirements of the codes are not always respected. The reasons of that can be different. The requirements of codes are not always in relation with the possibilities of the workshops, the control is not sufficient and, in spite of the strict control at the workshop, the imperfections can overstep the allowable limit when they are measured on the bridges subjected to their dead weight.

To establish realistic values of imperfections, the statistical approach is recommended. In part 3, the results of many hundreds of measurements about 14 bridges, effected in four european countries, are summarized.
As representative value of the establishment of fabrication tolerances, the 95 % fractile was accepted. This value should eventually be lowered according the fact that a low probability of failure has already been provided against variations in yield stress (see part 3). It would be useful to make a deeper analysis of this topic.

5.4.

It is recommended to unify the measurement procedures and control parameters in all countries in such a way that these measurements be simple to apply practically and represent simultaneously reliable and quantifiable measures of the strength of the elements of plate.
The majority of specialists recommend to take as relative imperfection the quantity f_1/b or f_2/a (fig. 2) because it is easy to apply. However, in special cases, it can be recommended to adopt the method utilized in United Kingdom with a gauge bar, gauge length 2b, in the longitudinal direction, in order to exclude a "hungry horse" component (fig. 6) which has no deleterious effect for the ultimate strength of plate (cf. sec. 3.2.1.).

5.5.

The number of measurements is not important enough and their range is so large that it is difficult to propose at the present time unified values of fabrication tolerances which could be acceptable for all the countries.
Nevertheless, it seems that, for the moment, it is possible to propose the following values (applied to unloaded bridge) which can generally be respected :

$$\frac{f_1}{b} = \frac{1}{200} \quad \text{(deformation of plate panels)}$$

$$\frac{f_2}{a} = \frac{1}{500} \quad \text{(deformation of longitudinal stiffeners)}$$

However, if a shorter gauge length ($G < a$, with a the length of the stiffener between cross members) is used in establishing the tolerance (as described in part 3) the actual out-of-straightness of the stiffener at mid-length will be greater than a/500. For the moment, in order to eliminate such interpretation, we propose that the gauge length must always be "a" and not shorter.

In countries where the measured imperfections are not in accordance with this proposal or in countries where there are no measurements, an economical study of the interaction strength-cost will have be done before adopting or changing above values of the tolerances.

5.6.

Residual stresses influence the behaviour of compressed plate panels. However, it is generally agreed that, after fabrication and assembling, the residual stress pattern is so complicated that it cannot be predicted with confidence. Therefore, the present Committee is of the opinion that no measurements of residual stresses should be imposed.

5.7.

It is recommended to continue on a large scale the measurement of imperfections and their statistical analysis in order to estimate the real correlation between measured, prescribed and effective values of tolerances which presently could be accepted in design. Parallel to this effort, it is recommended to develop a Reliability Assessment System as mentioned in section 1.2.

NOTATIONS

a	plate panel or longitudinal stiffener length
b	plate panel width
t	plate thickness
f_1	out-of-plane deviation of a plate panel (out-of-flatness)
f_2	out-of-plate deviation of a longitudinal stiffener (out-of-straightness)
f_3	out-of-plane deviation of a cross-girder or cross-frame
w	deflection at a point x, y
α	geometrical parameter $= \frac{a}{b}$
β	geometrical strength parameter $= \frac{b}{t} \sqrt{\frac{\sigma_r}{E}}$
$\beta^x = \frac{1}{1,9} \beta$	
σ_r	yield stress
G	gauge length
W_o	amplitude of an initial deflection
W	amplitude of an additional deflection
E	YOUNG's modulus.

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C.1 to C.13 References to the Codes (see chapter 2.2.).