Free discussion

Objekttyp: **Group**

Zeitschrift: **IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen**

Band (Jahr): **11 (1971)**

PDF erstellt am: **24.06.2024**

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Topic: **Box Girders, Hybrid Girders, Fatigue Problems, Special Problems, Concentrated Loads, Plates with Holes**

Poutres en caisson, poutres hybrides, problèmes de fatigue, problèmes spéciaux, charges concentrées, plaques avec trous

Kastenträger, Hybridträger, Ermüdungsprobleme, Sonderprobleme, konzentrierte Lasten, Platten mit Löchern

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PROF. K.C. ROCKEY Chairman's introductory remarks.

During this third and last working session, we shall be discussing the very important subject of the behaviour of box girders together with the special problems involved in the design of webs containing cut outs, the design of hybrid girders, the buckling and ultimate load behaviour of webs subjected to edge patch loading and the fatigue behaviour of plate girders.

These are all problems of considerable importance and since we will have much to discuss, we must of necessity endeavour to keep our comments to a minimum.

Each author was given the opportunity of briefly introducing his report and the following discussion ensued.

PROF. FUKUMOTO.

I would like to add a few comments with respect to the paper by Professor NISHINO and OKUMURA. I refer you to Page 19 where there is a brief description of the plate girder research and the first part is on the shear strength of welded built up girders and this report has already been presented in the final report of the 8th Congress of I.A.B.S.E. and Professors ROCKEY, BASLER and OSTAPENKO have already referred to this paper in the discussion of experimental results yesterday, so I am not going to present the first part of their report.

The second report is on the moment carrying capacity of large size rolled I beams with the depth of 900 mm and 300 mm in the flange width. They have three different kinds of beams. The first of their tests was carried out on "as rolled" I shapes and the second one on "annealed" I shapes and the third on cambered I shapes; the cambering is due to the stretching of the compression flange during the rolling process. The main aim of their study was to measure the major residual stresses in all 3 shapes and all three involve a different fabrication process. They noticed that the maximum magnitude on the residual stresses reaches 60 % and in some 90 % of the yield strength and they are distributed over the wider portion of the cross section. Due to presence of these large compression residual stresses in the web plate, there are a number of specimens in which the conditions in the web plates become critical for buckling. During the bending test of full size rolled beams, it was observed that the test beams behaved as if they were made of materials of different strength, which was due to the penetration of premature yielding at portions where larger magnitude of residual stress occurs. The fact indicates that a larger reduction of rigidity occurs even at relatively small loading conditions; however, the width to thickness ratios of component plates of rolled beams were so small that stability was not lost by the reduction of rigidity due to the premature yielding and as a consequence the moment carrying capacity exceeded full plastic moment. It was noticed that a beam with buckled web plates, prior to the application of external load, stabilized with the increase of bending moment.

PROF. MASSONNET.

Professor FUKUMOTO, I want to know whether you have special provisions in Japan for the design of box girders, a subject that we shall treat later on in this morning's session, because I was quite interested to see what you said about railway bridges, road bridges and ships.

PROF. FUKUMOTO.

I think that some parts of the Japanese specifications are like those of the German specifications. We do not have special specifications for box girders. We can deal with the design of the compression flange of the box girder as a stiffened plate. Since we do have a specification for the design of a stiffened plate girder, they apply also to stiffened plates under uniform compression. But we do not have special codes for the box girders.

PROF. MASSONNET.

Is this specification to which you refer one derived for ship structures or for compressed stiffened plates ?

PROF. FUKUMOTO.

Yes, we use the specification developed for ship design. They have been studying this problem. If there is time later this morning I will discuss the behaviour of compressed stiffened plates.

PROF. MASSONNET.

Yes, good, because in my opinion the behaviour of box girders depends very much, almost uniquely, about the behaviour of the compressed plate.

PROF. ROCKEY.

If I understood you correctly, Professor FUKUMOTO, you said that, if the web is initially deformed then you find that on loading the bending stresses tend to straighten the webs out. That is what SKALOUD and myself also experienced. Nowhere in the Conference today have we really mentioned the influence of residual stresses; there is for example, the paper by OWEN who measured the residual stresses in plate girders and it is clear that the presence of these stresses would be to reduce the buckling stress of the web. This is a consideration which none of us have brought into our previous discussions. This, for example, will mean that in the shear models the critical stress distribution which acts in the "triangular areas" will be reduced because of the presence of residual stresses. Have you any comments on this factor ?

PROF. FUKUMOTO.

Actually, for the large rolled section girder, there was an extremely high residual compressive stress in the web and, when the bending moment is applied in the tension area, you eliminate these high compression stresses and this stabilizes the web - that is one of the findings in their paper. In some cases, especially in bending, the initial compressive stresses in the web can result in a higher ultimate load.

PROF. OSTAPENKO.

It seems to me that buckling of the web will be promoted by the compression zone and restrained by the tension zone, so the findings do not appear to be quite logical.

PROF. FUKUMOTO.

Due to the presence of compressive residual stresses of large magnitude in web, the web plates are critical for buckling.

PROF. OSTAPENKO.

Also, residual stresses are usually considered in buckling computation indirectly by taking the material to be linear up to about 50 to 80 % of the yield stress and then introducing some kind of a transition curve up the yield analogous to a column buckling curve.

PROF. FUKUMOTO.

There are very high compressive stresses at the beginning and during the rolling of girders. We have already experienced the buckling of the web plate due to high compressive residual stresses.

PROF. OSTAPENKO.

Well, the web is never perfectly flat anyway.

PROF. FUKUMOTO.

And also, along the connection between the flange and web there are high tensile residual stresses and I am not sure which part of the web from the top of the flange does possess high compressive stresses. This high tensile residual stress along the flange-web weld line gives favourable results to the web buckling.

PROF. OSTAPENKO.

This may also be the function of the depth of the girder in inches since the width of the tensile residual stress is only about 2" which is usually a small portion of the total depth and the remainder is under compression.

PROF. FUKUMOTO.

Professor NISHINO computed the linear buckling of the plate with initial stresses and also they found good agreement of their computation values with the test results.

PROF. OSTAPENKO.

But that was for uniformly distributed strain.

PROF. FUKUMOTO.

No, for bending of the web.

PROF. DUBAS.

Do you have curves for buckling of the webs of rolled shapes ?

PROF. FUKUMOTO.

Yes. These are the curves for rolled shapes. As a result of rolling of the large size girders, we sometimes obtain a buckled web after the rolling process.

OWEN R. Welding Residual Stresses in plate girders. Civil Engineering TRANSACTIONS, Institute of Australian Engineers, Oct. 1969, p.157-161.

PROF. BEEDLE.

Are you referring to the inspection of the shape after it is rolled and the web is waving ?

PROF. FUKUMOTO.

Yes, for the large size panels of side 900 mm, one has a big problem in preventing the web buckling during the rolling process.

PROF. BEEDLE.

What depth to thickness ratio would you have ?

PROF. FUKUMOTO.

The limiting thickness ratio is 60 or 65 because of the initial buckling of the web dueto the rolling processes.

PROF. BEEDLE.

How are these cooled ?

PROF. FUKUMOTO.

It is during normal cooling on the cooling bed that they start to wave in their web.

PROF. BEEDLE.

But these are rejected, are they not ?

PROF. FUKUMOTO.

Yes, they are rejected.

PROF. BEEDLE.

So this is a development that you observed on the cooling bed ?

PROF. FUKUMOTO.

Yes.

PROF. BEEDLE.

Well, that shows a very high cooling rate I would guess and I would just like to make an observation. The very first evidence of residual stresses at Fritz Engineering Laboratory were in the 1930's when tests of box girders for crane girders were being carried out. These girders were welded and after the welding operation the compression flange plates, which they knew initially were straight, were observed to have waves in them, and this was the first time that this influence was observed and, of course, it leads to a lowering of the 'buckling stress' because the plate is already buckled before any external load is applied.

PROF. ROCKEY.

Thank you Professor BEEDLE. We are all very aware of the very valuable and interesting work which Professor LAMBERT TALL and his co-researchers have carried out at Lehigh. The work by OWEN (see foot note page 5) to which I referred was conducted on a deep plate girder web where he shows that these residual stresses can be significant.
Our next paper is by Mr. HÖGLUND. I found the paper by Mr. HÖGLUND very interesting since this is clearly a topic which is becoming increasingly important in building construction. I would like to ask Mr. HÖGLUND if he foresees any difficulties in applying his method to the case where you have holes spaced regularly along a beam. I would suspect that one would not presumably attempt to use this method in the analysis of that type of problem.

MR. HÖGLUND.

Well, I have not studied the problem of a plate with many holes, only the case where the distance between the holes is so large that one hole will have a very small influence on the web behaviour around other holes. Restrictions about the distance between the holes are given in my paper and I think the model could be extended to the case of many holes too, but this will need further research work.

PROF. MASSONNET.

I want to raise a very highly academic question.
You may know about the paper by MANSFIELD, a distinguished researcher of the Royal Aircraft Establishment, about neutral holes in plane stress. Suppose you take the fuselage of an aeroplane and you want to find the optimum window. I think this study arose from the accident of the Comet N° 2 - I think - that crashed after an explosion due to fatigue of the fuselage. MANSFIELD has solved the problem of the design of reinforced holes of such a shape that the sheet does not realise there is a hole in it, there is no stress concentration at all. You have the optimum size of the hole on one side and the optimum reinforcement around the hole on the other side. Now I would like to ask the same question for buckling. We could ask - it is highly academic of course - how to reinforce the opening so that the buckling stress of the web with a hole is just the same as that of a web without a hole.

MR. HÖGLUND.

My investigation is confined to the case of holes in unstiffened thin web.

PROF. ROCKEY.

Professor CHEUNG, Dr. ANDERSON and myself have provided this information in a recent paper and this work is currently being extended.

PROF. OSTAPENKO.

Mr. HÖGLUND, have you compared your results with BOWER's and REDWOOD's studies ? Some of their work has been published in A.S.C.E. Journals.

MR. HÖGLUND.

Yes, I have seen this work.

PROF. OSTAPENKO.

BOWER used the Theory of Elasticity with the limitations that the hole be at the mid-depth and its diameter should not exceed, I think, 60 % of the depth. I notice that you go up to 90 %. But how do your results compare with his ?

MR. HÖGLUND.

His tests were on girders with webs much thicker than those employed in my girders.

PROF. OSTAPENKO.

Yes, yes, I understand.

PROF. HÖGLUND.

Therefore we used BOWER's tests to see whether my theory for thin webs was applicable for girders with thick webs. My theories are ultimate load theories.

PROF. OSTAPENKO.

He made elastic studies and also considered plastification, i.e. ultimate capacity. Thus his results should apply to your case also for low depth-to-thickness ratios. He also established curves for designing holes and reinforcements.

DR. FLINT.

Who has done that work ?

PROF. OSTAPENKO.

Jack BOWER. He is at U.S. Steel.

PROF. BEEDLE.

Alex, would the unpublished work that KUSUDA did with Bruno (THÜRLIMANN) have any application to this problem ?

PROF. OSTAPENKO.

That was a formulation of a Vierendeel truss deformation mechanism for rectangular holes. They studied the reinforcement requirements needed to develop the plastic moment capacity of the original section.

PROF. BERGFELT.

You asked for investigations on webs with closely spaced holes (cf. page 7). A special case, castellated beams, has been investigated also in a thesis by Dr. R. TEPFERS, CTH, Göteborg. If you wish, I will ask him to send a copy to you and anyone else who is interested.

PROF. ROCKEY.

Thank you very much.

PROF. OSTAPENKO.

I would draw your attention to the fact that Professor REDWOOD at McGill University is also doing research on webs having closely spaced holes.

PROF. ROCKEY.

The next paper is the paper on box girders study by Professor DUBAS; perhaps Professor MASSONNET you could summarise this work in English.

PROF. MASSONNET.

Well, Professor DUBAS in his written report has considered the case of a box girder with longitudinal stiffeners having the theoretical optimum rigidity γ^*. That means stiffeners for which you can guess whether the stiffener will be bent or remain straight at the critical stress given by the linear theory. But, of course, all these stiffeners bend in the post critical range. Professor DUBAS, in his first test, used this type of stiffener and he has shown that there was a large pocket in the stress distribution. You have seen that the stresses at the edges of the box girder were at yield and the stresses in the middle were very much lower. Now, the second test specimen which has the reference N° 1 in his report, has the same proportions as the other one, except that the stiffeners have been increased to four times the theoretical γ^* rigidity. Then he has obtained a completely different picture with a stress distribution at failure only having a small general pocket. The increase of strength which is achieved is about 85 %. Now, the additional cost of steel is almost nothing because fabrication costs are the same in both cases. Professor DUBAS strongly recommends to use a 'm' factor of about 5 for multiplying

PROF. MASSONNET (continued)

the theoretical value of the optimum rigidity (γ^*). He has also tried to apply some of the formulae presented by Professor COOPER by considering one of the stiffeners as a bar subjected to column buckling and he reports that he obtained bad agreement in this case. He proposes to use the linear buckling theory but to increase the flexural stiffness of the stiffeners five times.

PROF. OSTAPENKO.

Did I understand correctly that Professor DUBAS includes 20 times the plate thickness to act with the stiffener ?

PROF. DUBAS.

Yes.

PROF. OSTAPENKO.

And also did you consider that the load was applied to an unsymmetrical stiffener section ?

PROF. DUBAS.

The beam is subjected to pure bending, so that the stress distribution is constant.

PROF. MASSONNET.

Professor OSTAPENKO means by his question; is the compression acting at the centroid ? Is that right ?

PROF. OSTAPENKO.

Actually, it is not applied to the centroid of the plate-stiffener combination - it is applied through the plate and therefore the stiffener would tend to bend away from the plate and thus weaken the whole panel.

PROF. DUBAS.

In the tested panel, the shear stresses between stiffener and plate are

PROF. DUBAS (continued).

theoretically zero, so that the total compression is acting in the centroid of the stiffened plate.

PROF. OSTAPENKO.

However, at the later stages of loading the effective width of the plate is reduced and the assembly behaves as a beam-column.

PROF. DUBAS.

Oh no, not in this case.

DR. FLINT.

I am in agreement with Professor DUBAS.
Professor MASSONNET, the question of effective width is important, I am interested in this comparison with the strut. As the plate between the stiffeners deforms, the effective stiffeness decreases until in fact it eventually vanishes altogether, when you reach yield at the junction. So that you should allow for a variable effective width as the strut buckles and of course this interaction goes on to complete failure. Have you tried doing this ?

PROF. DUBAS.

No. I compared the two test results and, for this relative behaviour, the value of the effective plate width is not very important.

PROF. COOPER.

Professor DUBAS says that the agreement with the column formula was poor. Does that mean that the column formula predicted a higher strength?

PROF. DUBAS.

Yes, this question is discussed at length in my report.

PROF. BEEDLE.

Professor DUBAS, I wonder if the difference there could not be the difference simply of post buckling strength that you observed in the web of a girder as compared with the behaviour of the flange acting as a column.

PROF. MASSONNET.

I want to say that if you look at the American specification A.I.S.I. developed by Professor George WINTER, you see that for the light gauge steel construction there is a large reserve in post buckling strength, even in box girders.

PROF. BEEDLE.

I would not say there is not any but I would simply state without making any studies at all, guess that the post buckling strength would be greater in the case of the longitudinal stiffener in the web than it would be in the cross flange of a box girder, so that I would expect that the influence of the residual stresses and so on, to be greater in this context than in the longitudinal stiffened web.

PROF. MASSONNET.

The only point that I want to make is that considering the stiffened compressed flange as an assembly of independent struts would be going backwards and to be unnecessarily safe.

DR. FLINT.

I do not think that you are necessarily going backwards in considering the assembly of the struts providing you consider those struts to have an increased Euler load which allows for the plate behaviour. You could use a fictitious strut, by using orthotropic plate calculation together with the effective width formula.

PROF. MASSONNET.

That is just the point I want to discuss in my paper.

PROF. ROCKEY.

Thank you Professor DUBAS for your contribution, we will come back to discuss this. Professor MASSONNET, after he presents his own contribution to the prepared discussion, is going to very kindly communicate to the body here, the views of Professor LEONHARDT and Professor KLÖPPEL, which have been expressed both to Professor MASSONNET and myself in private communication.

PROF. MASSONNET.

I wish to emphasise one of the main points contained in the paper by Mr. MAQUOI and myself.

You know what are strictly rigid stiffeners (defined by their theoretical relative rigidity $\gamma^{*}= \frac{EI_s}{lD}$.) They are the smallest stiffeners which the linear buckling theory predicts to remain straight under the critical stress when the plate is perfectly flat. This theory does not say anything about the behaviour of such stiffeners in the post buckling range, because, according to the linear theory, no such range does exist. Anyway, as practical plates are never flat, all experiments have shown that γ^* stiffeners invariably bend since the beginning of the loading.

I insist therefore very strongly on the point that, even for plate girders, the safety against collapse offered by such stiffeners is insufficient because the experiments show that the elastic collapse stress of my model girders so stiffened was only about 95 per cent of the yield stress of the material. We attach therefore a basic importance to the recommendation given in (1,2)*: for longitudinal stiffeners, take $\gamma = m\gamma^*$ with $m = 6$ to 8, and you will obtain an increase in ultimate strength of 20 to 25 %, with an increase in weight less than 5 % and an increase in price still smaller.

This result has been nicely confirmed and precised by a paper of OWEN, ROCKEY and SKALOUD (3).

The diagram Fig. 1, taken from this paper, shows that the benefit derived by adopting for the two longitudinal stiffeners, m = 7 instead of 1, is about 20 %.

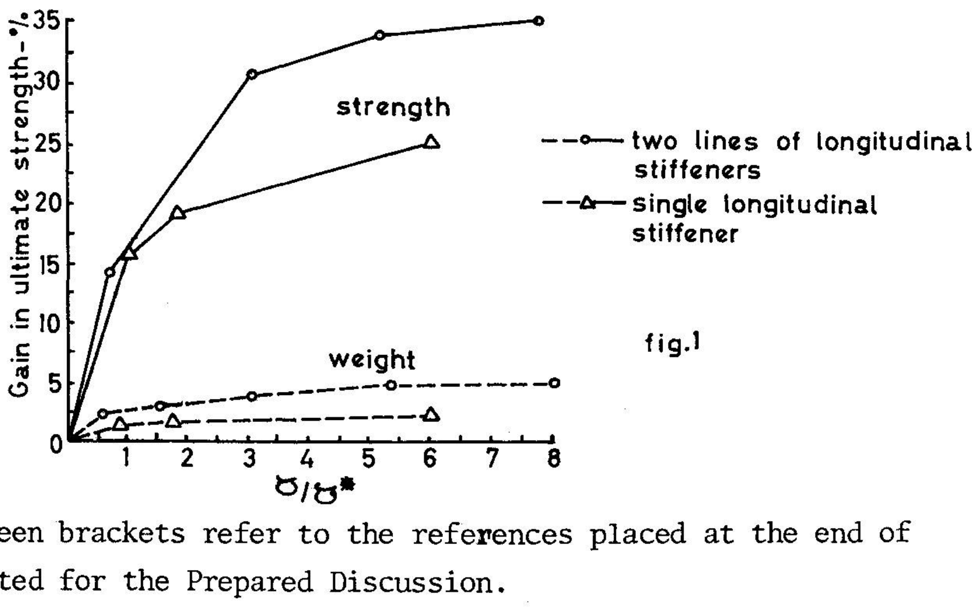

fig.1

* The numbers between brackets refer to the references placed at the end of the paper submitted for the Prepared Discussion.

PROF. MASSONNET (continued)

We are presently trying to develop a non linear theory for the ultimate strength of box girders. In our mind, the same factor of safety should be applied against this collapse strength as against yield for members subjected to tension or bending, namely 1.5 in the regular case and 1.33 for erection conditions.

Now that several accidents have taken place, we hear that we should calculate the stiffeners as compressed struts. We at Liège are against this viewpoint because it is oversafe to do so; you would waste much steel.
An ultimate strength theory of stiffened box girders must, we feel, take account of the following facts:

1) the initial deflection f_o of the stiffened panel ;
2) the eccentric position of the stiffeners. I have discussed the effect of this eccentricity in a paper published in the 1959 Volume of the "Publications of IABSE", entitled - Plaques et Coques à raidisseurs dissymétriques. My paper was based on a study made in Germany by Professor A. PFLÜGER.
3) the stiffeners are much too numerous to be considered individually.

Doing so would be wasting computer time. Like in the so-called GUYON-MASSONNET method for calculating slab and multiple beam bridges, we have to spread out the rigidities of the stiffeners continuously, but still taking account of their eccentric position. We have been able, these last two weeks, to generalise PFLÜGER's theory by adding to the classical expression of the strains $\varepsilon_x = \partial u/\partial x$, $\varepsilon_y = \partial v/\partial y$, etc.. the non linear second order terms $1/2\ (\partial w/\partial x)^2$ etc.. which are given by finite theory of elasticity and which have been introduced in theory of plates by Th. von KARMAN.
This generalised theory yields two coupled fourth order non linear partial differential equations: an equilibrium equation governing the transverse displacement w and a compatibility equation governing the AIRY stress function $\emptyset$ for the membrane stress state in our membrane plate.

Regarding the boundary conditions concerning w, we may assume simple support along the four edges (w = curvature = 0).
The boundary conditions concerning the AIRY stress function are more complicated: First, we may assume $N_{xy} = 0$ along the four edges (Fig. 2).

PROF.MASSONNET (continued).

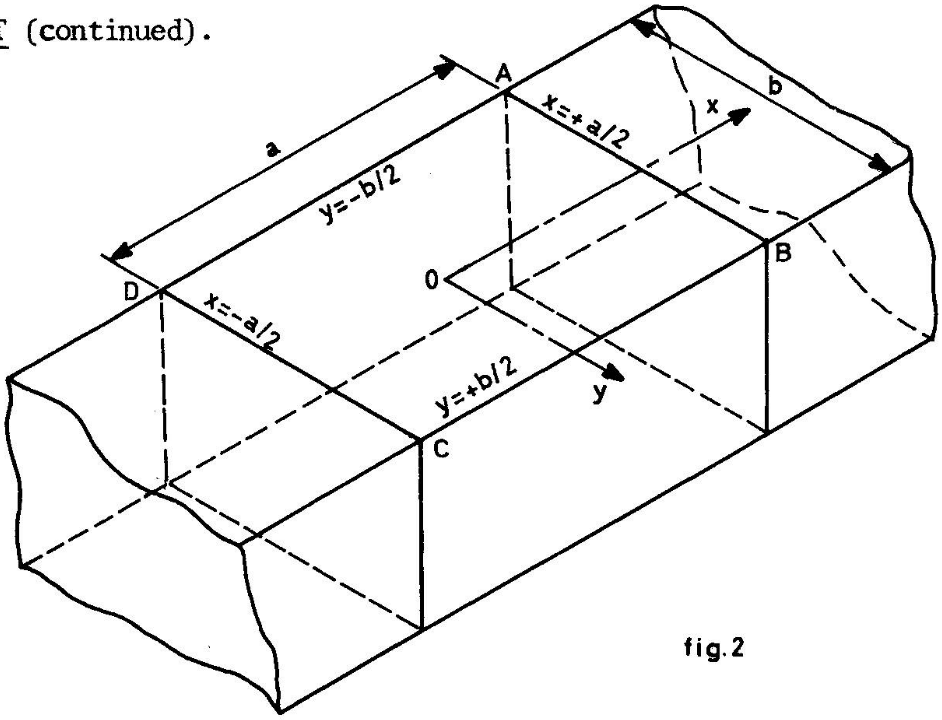

fig.2

Along the unloaded edges y = - b/2 and y = + b/2, N_y must be zero, because the webs of the box girder are very flexible normally to their plane.
Along the loaded edges, x = - a/2 and x = + a/2, we must express the condition that these edges are actually nodal lines, which remain straight by symmetry, even in the post critical domain. We then express mathematically the condition that the variation in distance between x = - a/2 and x = + a/2 is the same for all corresponding points, and therefore does not depend on y.
With the coordinate axes placed as indicated by fig. 2, we may for simplicity assume an initial deflection of the shape

$$w_o = f_o \cos \frac{\pi x}{a} \cos \frac{\pi y}{b}.$$

If we take for supplementary deflection the first buckling mode

$$w = f \cos \frac{\pi x}{a} \cos \frac{\pi y}{b},$$

we may integrate the compatibility equation rigorously in closed form, but it is impossible to integrate the equilibrium equation rigorously because above expression of w is too simple.
We then resort to the BUBNOV-GALERKIN technique, which gives the value of the amplitude f minimising the error throughout the rectangular field of integration ABCD.
We adopt the same collapse criterion as VOLMIR and SKALOUD, namely that collapse occurs when the mean membrane stress along the unloaded edges y = - b/2 and y = + b/2 reaches the yield stress.

PROF. MASSONNET (continued)

After lengthy calculations, we obtain an expression for the mean collapse stress $\overline{\sigma}_c$ and, in our opinion, box girder bridges should be designed so as to present under the (collapse) factored loads SP, with S equal to 1.5 or 1.33, an actual mean stress $\sigma_a < \overline{\sigma}_c$.

PROF. ROCKEY.

I thank our speakers who have presented very interesting and revealing comments with respect to the behaviour of the compressed flanges. I wonder if anyone wishes to raise any specific questions ?

DR. SKALOUD.

I would like to sum up in just three sentences the prepared discussion which I have prepared. The theoretical research is based on the non linear deflection and the experimental research which we have conducted at the Institute of Applied Mechanics at Prague has demonstrated that the currently held γ^* concept does not ensure that the stiffeners will remain effective in the post buckling range. I agree with Professor DUBAS and Professor MASSONNET that this concept should be abandoned as soon as possible in order that accidents like those mentioned by Professor MASSONNET should be avoided and I would like to take this opportunity to recommend to the Organising Committee that further research in this area, both theoretical and experimental should be included in the recommendations presented in the final report.

PROF. ROCKEY.

Thank you Dr. SKALOUD. Professor COOPER.

PROF. COOPER.

I am convinced, and I think perhaps others are also convinced, that a column analysis of the stiffener acting with a part of the web can be applied to this problem. Professor MASSONNET has pointed out that part of the problem of applying this type of analysis is to determine the effective width of the web plate to be used with the stiffeners. Another problem is: what are the end conditions of the column ? Anyway, it is clear that this stiffener-column problem can be solved, and I am a little bit puzzled as to why it has not been done successfully to date.

DR. FLINT.

There is another aspect to the column problem. If you apply the two modes of failure to the column, one case is that of reaching yield in the outstand which might be with the column buckling away from the outstand and the other is of course the collapse of the plate associated with the column. If however you buckle in the opposite direction, you have to consider what would be the exact conditions and you have to consider the composite column with a decreasing flange with a failure condition which is not the onset to yield - at the junction between the plate and the stiffener but is something less than that perhaps - something much less because you see you have got a rather complex problem. I think that a practical approach, using an equivalent Euler load to start with, to take into account the two dimensional nature of the system, would I think, be the way to deal with the problem in the first instance.

PROF. CLARK.

I think that in many practical cases the effective length would probably be the full length of the panel. It would only be for relatively long panels that you would have a shorter effective length.

PROF. MASSONNET.

Professor DUBAS has already demonstrated that it was not so.

PROF. DUBAS.

It is perhaps the effective length. Oui, mais la longueur de flambement d'une poutre dans un milieu élastique, ce n'est pas la longueur réelle.

PROF. BEEDLE.

You are saying that the points of lateral strength did not correspond to the points of inflectionbut instead you had a longer effective length of the panel.

PROF. DUBAS.

Yes, yes. In a pony truss, the effective length is not the sine length, because you have the reaction of the medium.

PROF. BEEDLE.

Flexibility of the supports ?

PROF. MASSONNET.

The second figure by Professor DUBAS has shown that the wave lengths in the longitudinal direction as well as that in the transverse direction are very short when the compressed plate is effectively stiffened. Five waves being formed in the longitudinal direction and 4 in the transverse direction, i.e. the development of small longitudinal buckles, like those I have shown in the photograph of the damaged sections of the Vienna Bridge.

PROF. COOPER.

Did you not have transverse stiffeners ?

PROF. DUBAS.

Only one transverse stiffener.

DR. FLINT.

But this is what I mean by an effective Euler load, that you take into account that wave length which will come out on the orthotropic plate theory.

PROF. GACHON.

Le problème est encore plus compliqué, parce que le raidisseur a un comportement élastique mais non linéaire.

PROF. ROCKEY.

I wonder if Professor MASSONNET could very briefly communicate to us the views of Professor LEONHARDT and Professor KLÖPPEL.

PROF. MASSONNET.

Well you know, Professor LEONHARDT, although unable to attend the Colloquium, readily agreed to explain in broad terms the procedure of his consulting office in checking buckling stability. He sent me a letter on February 22nd. I shall read you the second part of the letter rather slowly.
(see corresponding text on pages 429).

PROF. ROCKEY.

Does anyone wish to comment on this letter from Professor LEONHARDT ?

PROF. MASSONNET.

This is an expression of the point of view of a practical man engaged in bridge design.

DR. SKALOUD.

Does Professor LEONHARDT make any recommendations for determining stiffener proportions ?

PROF. MASSONNET.

He told me that he tried to verify them as struts. On this point I do not agree with him, but I believe it to be a safe procedure.

PROF. COOPER.

It seems to me that the transverse diaphragms for the box girder should be proportioned such that they could be considered rigid in their support of the longitudinal stiffeners and that the other intermediate transverse stiffeners between diaphragms would be there to get orthotropic action.

PROF. MASSONNET.

I think that the opinion of Professor KLÖPPEL will perhaps clarify a little more the situation. It is not my own contribution but it just presents the views of Professor KLÖPPEL.

Professor KLÖPPEL sent a letter to Professor ROCKEY on March 2nd and said that he could not come and he said -- it is in German, I shall try to translate in English. 'Maybe I can draw your attention on the problem of the design of " "box girders, which because of these accidents is now very much up to date," and he refers us to pages 13, 14 and 15 of the second volume of his book "Beulwerte ausgesteifter Rechteckplatten" written by himself and Mr. MÖLLER. I received this notice only last Tuesday and I hurried to make a rather poor translation from German into English. On page 13 he says: (see corresponding text on pages 425 to 427).

DR. FLINT.

Can I ask a question. Is μ_k against a theoretical critical collapse or is it against a reduced collapse calculation ?

PROF. MASSONNET.

The German Specification DIN 4114 for short struts is based on the JAEGER theory which concerns an elastic -- completely plastic strut, with an initial eccentricity $e = \frac{i}{20} + \frac{l}{500}$The safety factor is $\mu_k = 1.5$ against collapse; then, for long struts you adopt the Euler theory and you take a safety factor μ_k of 2.5.

PROF. ROCKEY.

I am sure we are all very grateful to Professor MASSONNET for the time he has taken to translate it for those of us who are poor linguists and also for presenting these two communications so well.

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