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

Band: 11 (1971)

Artikel: General analytical methods - ultimate strength of plate girders subjected to bending and to shear and bending

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DOI: https://doi.org/10.5169/seals-12071

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Topic: **General Analytical Methods**
Ultimate Strength of Plate Girders Subjected to Bending and to Shear and Bending

Méthodes d'analyse générales
Résistance à la ruine des poutres à âme pleine soumises à la flexion avec cisaillement

Allgemeine Berechnungsmethoden
Traglast von Vollwandträgern unter Biegung und Querkraft

Chairman: **CH. MASSONNET**
Professor
University of Liège
Belgique

PROF. C. MASSONNET   Chairman's introductory remarks.

This afternoon we shall be discussing the theoretical papers by Professors Gachon, Burgermeister and Steup who have examined the post buckled behaviour of elastic plates. In addition we have contributions from the American, Japanese and British representatives presenting their models for the Collapse behaviour of plate girders subjected to shear and bending.

Finally, we have the paper by Professor Osipov examining the effect of initial distortions on the behaviour of welded plate girders. In view of the wide range of topics under consideration it will be essential for all delegates to be as concise as possible.

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

PROF. C. MASSONNET.

I note that Professor Gachon states that he has observed that not only were the tensile stresses in the post buckling range larger than the critical buckling values, a fact known to everyone, but that in addition he has observed that the compressive stresses were also increasing up to one and a half times the critical load and I would like to ask whether anybody else has observed this.

PROF. O. STEINHARDT.

As I showed in the first slide (compare Figure 10 of my paper), you could see that, by increasing the load, the shearing stresses are distributed in the same way as before; it has been found that if there are even four or more (elastic) buckles in the tensile stress field, this distribution does not change significantly until the final stage. Later on, the diagonal tensile stresses were greater than the compressive ones.
As regards buckling of the web, you can obtain the corresponding information from the book by Wolmir*.

PROF. C. MASSONNET.

I know the book and we use it extensively.

PROF. O. STEINHARDT.

In this figure, a diagram is shown; In this case these are more buckles, and on the other hand there are shearing forces as before, and the rigidity modulus is only reduced by 6%; if the edges are laterally fixed, this modulus can be ultimately reduced by 15%.

PROF. C. MASSONNET.

This is a rather important question, because all of the ultimate strength models originating from Basler's model are based, more or less, on the assumption that the critical compressive stress remains equal to the critical stress deduced from the linear buckling theory and that only the tensile stresses increase up to the yield point. I would like to know your opinion regarding this question.

PROF. O. STEINHARDT.

In our tests, the elastic limit load is very high, it is nearly ultimate load, which is a plastic one. It can be recognised in Figure 4 of my paper.

PROF. C. MASSONNET.

Because I am conducting similar studies, I know that it is very difficult to develop a large finite element computing programme like the one presented by Professor Gachon. My question is: have you already obtained some numerical results?

* WOLMIR, A.S. : Biegsame Platten und Schalen

PROF. H. GACHON.

Pas encore. Les seuls éléments de comparaison qu'on a obtenus, c'est en linéaire sur des plaques orthotropes et en non linéaire sur des plaques non raidies. Lorsqu'on introduit les raidisseurs, on a actuellement des difficultés; il y a quelque chose qui ne va pas encore dans le programme à ce niveau-là.

PROF. P. DUBAS.

Avec votre programme, pouvez-vous déterminer la charge linéaire et la charge ultime?

PROF. H. GACHON.

Non, la charge réversible. La charge ultime c'est impossible, à moins de travailler pas à pas.

PROF. C. MASSONNET.

I will say that in English. If you introduce a linear criterion like that used by Dr. Skaloud, namely that on certain edges you come to the yield stress, then, I suppose that by your elastic finite element method you could go up to that level and consider that this level is the ultimate strength. But if you introduce the plastic deformation into the finite element, the solution becomes enormously complicated.

DR. A. FLINT.

Could I ask Professor Massonnet, was this work purely experimental or a combination of theory and experiment.

PROF. C. MASSONNET.

Purely experimental.

DR. A. FLINT.

I see, entirely experimental tests, I had hoped otherwise.

PROF. C. MASSONNET.

Professor Gachon has mentioned in answering my questions that they first developed the finite element programme for the linear case for orthotropic plates and then employed the non-linear Von Karman Marguerre equations, but only for isotropic plate.

PROF. H. GACHON.

Le programme était complet, il a été établi pour prendre en compte les raidisseurs, mais il ne marche pas encore dans ce cas.

PROF. A. OSTAPENKO.

My question is with reference to the load deflection plot obtained from the experimental results. The load coordinate indicates a critical load and a yield load. What boundary conditions were assumed in obtaining the critical load? I understand it was obtained theoretically.

PROF. C. MASSONNET.

This is the critical stress of the linear theory assuming the plate to be simply supported.

PROF. A. OSTAPENKO.

The web in a real girder, however, is restrained elastically. As an approximation, one may assume it to be fixed at the flanges and simply supported at the stiffeners. Then, the critical stress would be about 40 to 60% higher than that for a simply supported plate. Since the load deflection curve terminates just a little above this new buckling point and it still is quite steep, it seems that the plate has only started going into the post-buckling range. Thus, it should not be surprising that the compressive principal stress has deviated so little from the tensile principal stress. It is only later, at the loads beyond the final value shown that the curve should gradually level off to reach the ultimate load after developing a tension field. It is in this range where the difference between the two principal stresses should be quite pronounced with the compressive stress having about the same value as at buckling.

PROF. C. MASSONNET.

In other words, the good increase in compressive and tensile stresses observed by Professor Gachon belongs to the subcritical range, if the actual boundary conditions of the web at the flanges and stiffeners are taken into account.

PROF. H. GACHON.

Il est important de dire qu'on avait au seuil d'élasticité une charge de 18 tonnes, et ici de 30 tonnes, et qu'à ce niveau -

PROF. C. GACHON.
continued

là on avait encore des déformations qui étaient comparables aux premières.

DR. M. SKALOUD.

I would like to come back to the remark concerning Wolmir's book. I think that Wolmir's book deals with web panels attached to rigid boundary elements. Wolmir, using the non-linear theory of large deflections, gives a number of very interesting solutions; however, he does not allow for the flexibility of the flanges.

PROF. O. STEINHARDT.

Wolmir's theory is not confined to rigid flanges; they can have deformations in the web plane.

DR. M. SKALOUD.

Dr. Wolmir's theory accounts for the movement of the flanges towards each other, the flanges remaining straight. But it does not allow for the deformation or flexibility of the flanges, and just for that reason, his solutions are so simple. That is the first comment I wish to make and then I would like to ask Professor Gachon the following two questions. Quelle était la minceur de l'âme et quelle était la rigidité flexionnelle de vos raidisseurs verticaux?

PROF. H. GACHON.

Les âmes, 3 mm d'épaisseur, 700 mm de haut. Les raidisseurs, dans le cas des panneaux: membrures 300 mm de large et 10 mm d'épaisseur.

DR. M. SKALOUD.

Je suis assez surpris que le rapport des tensions principales soit proche de l'unité.

PROF. C. MASSONNET.

Dr. Skaloud says that the result obtained by Professor Gachon, namely that the tensile and compressive strengths are nearly equal, is not compatible with the experimental results he has obtained himself.

PROF. H. GACHON.

Je crois qu'on peut dire que tout le problème est de savoir à quel moment on fixe l'amorce du mécanisme.

PROF. C. MASSONNET.

I thank Professor Gachon for his presentation and I pass to Professor Fukumoto.

PROF. Y. FUKUMOTO.

Since the lateral strength of the compression flange is one of the main problems of girders in bending, our research was focused on this problem. These girders were designed and tested where web buckling occurred in the compression flange. A test set-up for a beam is shown in Fig. 2.2.2. The test beam is under uniform bending if the two jacks apply the same load capacity. No lateral bracings are provided for the test beam except at the loading points. The test beam is fixed to the loading beam through high strength bolts, and thus the boundary condition of the test beam is clamped laterally and torsionally at both ends. Each girder has two or three sub-panels in the web. Fig. 2.2.4. shows a typical example of load deformation curves for vertical, horizontal displacements and angular rotation at the compression flange of the span centre. The numbers in the figure correspond to the loading stages and the first one (0) is zero loading. At the second stage (1), the web plate starts to buckle and at the third (3), the girder reaches almost to the maximum point. The elastic straight line for vertical deflection is obtained from the elementary beam theory. The relative deflection of the web starts from the initial distortions of the web, and with the load increased the web buckled shapes become large to continue until lateral collapse occurs in the compression flange at the maximum moment. As can be seen from this figure, the web buckled pattern in each panel become obvious with the increased loads. In the central panel the direction of the relative web deflection is opposite to the lateral displacement of the compression flange, and thus the lateral displacement of the flange becomes small compared to the one in the side panel due to the resultant lateral forces transmitted to the flange along the web-flange weld line. Fig.2.2.9 shows the simultaneous buckling strength curves of the girders which combined web buckling, flange torsional and lateral buckling. Inelastic lateral buckling strength curves are also shown with the test results. Lateral collapse of the test girders in bending may occur when the moments reach the strength estimated by the inelastic buckling theory including residual stresses.

PROF. Y. FUKUMOTO.

continued

However, overall buckling curves may not give adequate estimation of the problem when the web buckles in several half waves.

PROF. C. MASSONNET.

Thank you very much, Professor Fukumoto. May I ask you whether one of your main conclusions is that linear buckling theory does not provide the failure load. You use a model accepting the linear theory of buckling of the plate as well as membrane effects and you say that you can ignore the buckling of the web and that the real damage is lateral buckling.

PROF. Y. FUKUMOTO.

Comparisons between the test results and reference moments such as buckling moment of an isolated web panel, overall buckling moment of the girders and lateral buckling moments indicate that the instability of the web plate does not give the proper estimation of the problem, in respect of the ability of the compression flange to resist lateral deflection. The ultimate strength is explained from the theoretical results for lateral buckling.

PROF. A. OSTAPENKO.

Does this mean lateral buckling of the compression flange alone or the rotation of the whole cross section?

PROF. Y. FUKUMOTO.

Yes, it is, I think, due to the lateral buckling of the compression flange. As I have shown in Fig. 2.2.4 of my paper, the distortion of the web plate first occurs as a result of buckling of the web plate panels, and, consequently, this leads to the lateral movement of the compression flange.

PROF. A. OSTAPENKO.

This means that the compression flange acts as a column?

PROF. Y. FUKUMOTO.

The compression flange also rotates in order to fulfil the compatibility condition along the web-flange weld line.

PROF. C. MASSONNET.

You imagine that the whole cross section is more or less maintained and that the rotation is general rotation of the whole section?

PROF. Y. FUKUMOTO.

Firstly, I have calculated the inelastic lateral buckling of the girders with residual stress distributions in which the rotation of the whole cross section was considered by neglecting the web distortion. However, in the inelastic buckling range, the results which were obtained by neglecting the St. Venant torsional rigidity are very close to the ones including the St. Venant torsional rigidity.

PROF. A. OSTAPENKO.

Yes, but if you neglect St. Venant's torsion when you have, say, Delta flanges possessing a high torsional rigidity, you will omit a considerable contribution to the buckling strength.

PROF. Y. FUKUMOTO.

In the case of the flanges of high torsional rigidity, lateral buckling phenomena of the girder would be different from the ordinary I shaped girders. And the torsional rigidity has to be taken into account in the analysis.

PROF. A. OSTAPENKO.

As Basler's tests on girders with 'pipe' flanges show, the flange tends to deflect sidewise without rotation, but this is accompanied by the distortion of the girder cross section.

PROF. Y. FUKUMOTO.

The buckling theory includes the weak axis flexural rigidity and also the St. Venant rigidity. However, in the inelastic buckling of the girders the effect of the weak axis flexural rigidity is predominantly high compared to the St. Venant torsional rigidity.

PROF. A. OSTAPENKO.

Yes, but interaction between torsion and weak axis bending for a cross section which is supposed to retain its shape may lead to a higher lateral-torsional buckling moment than one governed by the lateral buckling of the flange column alone.

PROF. Y. FUKUMOTO.

There is something which troubles me, because if we consider the flange at the time of lateral buckling, we have some stress concentration in the compression flange against the fibre stress calculated by conventional beam theory. This is due to the redistribution of the bending stress in the post buckling range

PROF. Y. FUKUMOTO.
continued
of the web plate.

PROF. C. MASSONNET.

Well I am afraid we have to move on - Professor Maeda; We have to look at the new paper which you have just presented. Professor Maeda has presented some interesting results particularly in the field of fatigue and on the influence of the flexural rigidity of a longitudinal stiffener on the fatigue strength.

PROF. K.C. ROCKEY.

Yes, I agree, I think this is a most interesting report. The finding of this report, like those of Professor Dubas from his work on box girders, indicate the significant influence of the effect of stiffener rigidity. Dr. Skaloud and I carried out a number of pure bending on plate girders with webs reinforced by longitudinal stiffeners. We found that unless we employed a stiffener having a rigidity corresponding to that obtained with the Massonnet factor of m of 4 to 8, the value varying according to the position of the stiffener, then we failed to achieve the full ultimate strength of the girder. This was because the longitudinal stiffener was deflecting as a beam column, with corresponding large web deformations. It would appear from these static tests and the fatigue tests of Professor Maeda that unless you employ a stiffener with a sufficiently high flexural rigidity so that it virtually does not deflect, then you will get high web deflections and this 'breathing' of the web will lead to the development of early fatigue cracking.

PROF. Y. MAEDA.

The important thing is that the longitudinal stiffener is welded to the transverse stiffener.

PROF. K.C. ROCKEY.

Agreed, however it does seem that we have established the need of using longitudinal stiffeners with high 'm' value both with respect to static strength and fatigue life. This is very interesting.

PROF. O. STEINHARDT.

From a special point of view, you can take high tensile

PROF. O. STEINHARDT,
continued

bolt-connections in order to obtain a limited friction point. Then you have a definite load for the longitudinal stiffeners. You can analyse beforehand the stress in the stiffener so that this latter does not buckle as a result of the connection moving at a definite point.

PROF. A. BERGFELT.

The area of the longitudinal stiffeners (or in reality the cost of these stiffeners) could be redistributed and be used to increase the thickness of the web or perhaps partly the thickness of the compression flange.

PROF. A. OSTAPENKO.

I am trying to see the significance in terms of dimensions since the buckling strength will be increased considerably by placing the longitudinal stiffener and this should be in an inverse proportion to the slenderness ratio. The question is then what portion of the area should go to the compression flange.

PROF. C. MASSONNET.

If I can say a word - I think that perhaps you are mixing two different questions. The first one is to analyse a definite type of plate girder and this is the problem which Professor Ostapenko has discussed. Now, another problem will be the most satisfactory girder from the financial point of view and we cannot say much about that.

PROF. A. BERGFELT.

You are quite right, and my intention was just to draw attention to the fact that there are <u>two different points</u> of view. The economical point of view is just the reason why I have done so much work on girders <u>without</u> intermediate stiffeners, especially for roof girders as reported in my contribution to this colloquium.

PROF. A. OSTAPENKO.

Perhaps I can give you half an answer. The increase in the case of shear or bending was rather small, but was quite dramatic under combined loads. Intuitively I feel that this redistribution of the stiffener area would not account for this effect in adequate measure.

PROF. P. DUBAS.

Pour un pont construit récemment en Suisse, on a dû remettre après montage un raidisseur longitudinal, parce que les déflexions de l'âme étaient trop importantes déjà sous le poids propre seul.

PROF. C. MASSONNET.

Professor Dubas mentioned a bridge in Switzerland, which initially had such large lateral deformations of the web that a stiffener was needed for deformation considerations.

PROF. L. BEEDLE.

What was it about the deformation that would require installation of the stiffeners?

PROF. P. DUBAS.

Je pense qu'il y a en plus des risques de fatigue très importants.

PROF. P. COOPER.

It is possible that you might want to use longitudinal stiffeners instead of increasing flange area because of fatigue. Where you have longitudinal stiffeners, you have an increase in static strength, but you need the stiffeners primarily to control the web deformations.

PROF. C. MASSONNET.

This is also my opinion and this is supported I think by the tests of Professor Maeda. So I believe there is a direct connection between longitudinal stiffeners and the fatigue strength of a girder. Anyway, even so, because of very high salaries paid to fabricators, you may want to simplify the design of the girder. Is that not so?

PROF. P. COOPER.

In other words, you want to use a slender web to save web area, then you might have to use a longitudinal stiffener to control lateral web deflections. However, this might be more expensive than using a heavier web.

PROF. P. DUBAS.

Dans le cas précité, ce n'est pas la fatigue qui a joué le rôle déterminant, l'âme étant en service encastrée par la dalle en béton. Il s'agit de serviceabilité: on ne peut livrer au maître de l'oeuvre un ouvrage tellement déformé.

PROF. C. MASSONNET.

These deformations took place before the completion of the bridge?

PROF. P. DUBAS.

Yes, during the construction of the bridge.

PROF. S. KOMATSU.

In my paper, the longitudinal stiffener bent upwards because of the lateral bending imposed on the longitudinal stiffener by the membrane action of web plate. I ask you the following question. Was there any phenomenon like this in the experiments by yourself?

PROF. C. MASSONNET.

You mean lateral bending? You mean that your photograph shows the bending of the longitudinal stiffener in the plane of the plate girder? That is moving up.

PROF. L. BEEDLE.

Well, the question is this: What moved? Did the stiffener move up? or the right hand move down?

PROF. S. KOMATSU.

I mean the longitudinal stiffener moves up at the midspan. The phenomenon seems to be induced by membrane action in the plane of web plate due to the influence of local bending of upper compression panel.

PROF. L. BEEDLE.

Well, has that stiffener failed?

PROF. S. KOMATSU.

Yes, it was plastically bent. So the tubular stiffener shown by Professor Massonnet needs to be used for preventing this sort of phenomenon.

DR. A. FLINT.

If we consider Professor Rockey's model, then at the junction of a transverse stiffener with the compression flange, to the left hand side of the stiffener one has a diagonal tensile field - and in the web on the immediate right hand side of the stiffener one is said to have only the critical stress acting. How then does this diagonal membrane stress get distributed into the vertical stiffener and the flange.

PROF. K.C. ROCKEY.

One has to depend upon the vertical (transverse) stiffener to transfer the vertical load and the flange the horizontal component at the junction of the web with the flange. As soon as one moves away from the flange, it can be assumed that the 'wedge' of web material to the right hand side of the stiffener will assist the stiffener in resisting the lateral bowing action imposed by the tensile membrane field. This stress distribution is complex and deserves a full study.

DR. A. FLINT.

Yes, and the question is, should there be another term in the strength equation which allows for the inplane bending stiffness of the vertical stiffener? In other words, if you had a stiff vertical stiffener in bending, as well as compression, would this also improve the load carrying capacity?

PROF. K.C. ROCKEY.

Well, one suspects so and we are back to the point that I was raising this morning, which is that insufficient work has been done to study such details. We really need to know more accurately than we do at the moment, the influence of the strength of both horizontal and vertical stiffeners and their influence upon the load carrying capacity of girders.
If I may I would like to now compliment Professor Ostapenko and his colleagues at Lehigh, I think they have done a very fine job and in fact, have continued the good work set up originally by Thürlimann and Dr. Basler. The essential difference between the Lehigh approach and the approach by Dr. Skaloud and myself is that we allow for the influence of the flange rigidity upon the width of tensile field whereas they employ a constant width band and assume it anchored against the vertical stiffener. The Lehigh researchers have also developed good design formulae for the ultimate design against lateral buckling of flange and stiffeners. I have not attempted to provide formula for compression buckling in flanges and would rely upon the use of a Massonnet Factor 'm' to design the stiffeners. Returning to the basic models, we are also differing on whether the flanges either provide a clamped support or simply support the web.

PROF. A. OSTAPENKO.

Yes, only I would like to extend credit also to Dr. Fujii and Professor Komatsu who offered their models. Actually I think the best model is probably some combination of all those proposed.

PROF. K.C. ROCKEY.

I would agree fully with this.

PROF. A. OSTAPENKO.

Frankly, the only advantage of our model I see at the moment over the other models is its ability to consider unsymmetrical girders in a very simple manner. In your case, you have to trade the upper flange against the bottom flange. The same applies to Fujii's model as well as to Komatsu's if they are extended to unsymmetrical girders. Actually, I think Komatsu's model is the most sophisticated one with respect to the distribution of the tension field forces in a panel under shear.

PROF. S. KOMATSU.

I do not think the tension field and stress redistribution in my theory is sophisticated, because these patterns were derived from the condition of strain compatibility in web plate. On the other hand, Professor Ostapenko's pattern of tension field was only assumed by intuition without any reasonable basis.

PROF. K.C. ROCKEY.

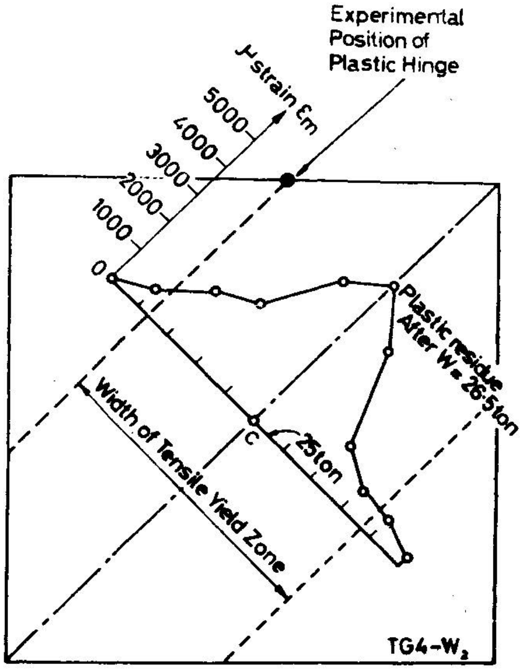
I would like to add a few comments with respect to the stress field which Dr. Skaloud and I have employed in our shear model. We have a diagonal band which is fully stressed to the yield value together with two adjacent wedges of web in which the stress is assumed to remain at the critical stress. Obviously there is not an abrupt change of stress at the junction of these triangular areas with the diagonal band. However, we have found that in most cases the tensile membrane stresses quickly decay away on either side of the diagonal band and for that reason we have adopted our rather simple model.

PROF. A. OSTAPENKO.

Initially, we tried to develop a pattern with the stress constant over the tension band and then some curve with a finite intensity at the flanges. As a trial we simplified to a linear wedge -- it gave very good correlation with test results, and we stopped at that.

PROF. K.C. ROCKEY.

The test results obtained by Dr. Skaloud and myself have indicated that there is a rapid change in the strains occurring in the neighbourhood of the two boundaries between the diagonal tensile membrane band and the adjacent "triangular wedges" in which the stress is assumed to remain at the critical buckling value. The rapid change in the strain is clearly demonstrated in the figure shown, which gives the diagonal strains occurring in girder.



PROF. P. COOPER.

Professor Rockey, I have a question -- are the strains you have shown, principal strains or just the strain in the direction of the diagonal.

PROF. K.C. ROCKEY.

They were diagonal strains, not necessarily principal strains.

PROF. P. COOPER.

It might be helpful in deciding which model best fits the experiments if the direction of the principal stresses were measured.

PROF. A. OSTAPENKO.

The answer to this question is that one group of tests will prove one model, some other group of tests will prove another model.

PROF. K.C. ROCKEY.

I believe Dr. Skaloud has measured the principal stresses on some of the tests we did and we found that they were reasonably close to the diagonal direction; but I accept your point.

PROF. A. OSTAPENKO.

Koni (Basler), you also measured principal stresses.

DR. K. BASLER.

Yes, actually we hoped to use this information but we noted the stress varied throughout the test. We always noted that with slender flanges the ultimate load was reached and then the deformation of the web increased and the web pulled the flange down, it was not so much a flange mechanism that constitutes the failure. I never thought of using a model involving the tension field action of the flange acting as a beam. I noticed that with girders of high aspect ratio of 3, the ultimate load tends to exceed the theoretical values obtained when assuming a simply support web and I knew that introducing clamped edges along the flanges one could accommodate the results. This I did not want to do because I had the development of the specifications in mind.

PROF. A. OSTAPENKO.

You could not have completed everything -- we would have been left without work.

PROF. L. BEEDLE.

Are you saying that taking into account the flange action is academic?

DR. K. BASLER.

You know I do not very much like the lower bound theory and what actually happens when you add bending moment on the shear, it has been pointed out this morning that we might only have one panel out of many in which we have shear alone.

PROF. L. BEEDLE.

Would that be, Konrad, because those flanges were relatively effective?

DR. K. BASLER.

Well, I should say that, despite the fact that the flanges were relatively flexible, we could have this remarkable observation that interaction actually does not take place as far as we observed.

PROF. C. MASSONNET.

Would you say, in other words, that there are two separate stages and the first stage is what you are describing and that there is another stage farther on with the development of a plastic hinge in the flange?

PROF. L. BEEDLE.

Yes. If we are sensible, we should not take advantage of this in design, or at least I would not.

DR. K. BASLER.

Well, on the average of the test results, we have some higher ultimate loads if we take into account flange rigidity. It is remarkable.

PROF. L. BEEDLE.

But I do not think the results have fallen short yet, have they?

PROF. A. OSTAPENKO.

An interaction curve for a transversely stiffened girder without a longitudinal stiffener has a similar appearance -- a relatively small, but greater than that for a longitudinally stiffened girder, reduction of the shear capacity due to moment and a small reduction of the moment due to shear in the right part of the diagram. For some other panels the reductions in both regions may be much more substantial.

DR. A. FLINT.

If I could ask a question, Professor Ostapenko. In most of the girders you tested, the web is very thin and having a small area in relation to the flange that would affect the shape of the interaction flange I suppose. The contribution of the web to the moment is small anyway, is that right?

PROF. A. OSTAPENKO.

Comparison of the theory with tests was made for girders with the web slenderness ratio as low as 50, that is, compact beams suitable for plastic design, to as high as 400!

DR. A. FLINT.

That curve would depend on how much of the moment capacity could be in the web to that in the flange, doesn't it?

PROF. A. OSTAPENKO.

Reduction of the moment capacity is rather small as indicated by the interaction diagrams, but the reduction of the shear capacity due to moment was quite drastic for this test series. These are interaction diagrams for individual tested panels.

PROF. A. OSTAPENKO.
continued

(See the inner diagrams in Fig. 12 of A. Ostapenko and C. Chern's report).

DR. K. BASLER.

Well, we should point out that these predictions are based on the measured propoerties of the girder material and measured shear. The yield strength not only depends on the material strength but from the orientation of specimens in the plate from which it cut. These properties and the tensile strength as introduced into your model already deviate from each other by as much as 15%.

PROF. A. OSTAPENKO.

Because of all these deviations from the nominal conditions, including the presence of residual stresses, we did not see the need for further refinements and, I assume, many others have felt the same.

DR. K. BASLER.

You have a somewhat higher ultimate load capacity than I used before because of the frame action, and then you let it drop. I would rather start at a lower value and keep it constant.

PROF. A. OSTAPENKO.

I have no detailed information here, but let us consider pure shear and plot the ultimate strength for each theory with various contributions separated with respect, say, to the web slenderness ratio and keeping other parameters constant, we find the higher ultimate strength given by one or the other theory alternately for different ranges of the slenderness ratio.
But, for example, the buckling contribution from our theory will always be substantially higher than from yours. Yet, the totals will be for many panels about the same. This really should be so since in the final analysis it's the yielding of the tension diagonal that is the criterion of the strength. Actually, your modification to direct the diagonal through the corners indirectly compensates for neglecting the flange contribution.

DR. K. BASLER.

Not only the model used to derive the bending or shear strengths but also the one for a combination is based on the

DR. K. BASLER.

continued

lower bound theory of plastic analysis; whenever we have a bending moment and shear, the flanges are assigned to carry the moment and the web carries the shear.

PROF. A. OSTAPENKO.

In our case, there is a continuous interaction between bending and shear for web buckling and then there are two strength branches -- shear reduced by bending and bending reduced by shear. The bending strength is controlled by the buckling of the compression flange column under the axial load from the moment causing buckling plus the axial loads due to the additional moment and due to the incomplete tension field force. The column section consists of the flange and a portion of the web - we used your value of 30t as a starting point but made it dependent on the yield stress.

DR. M. SKALOUD.

I would like to comment on the behaviour of webs in shear. I think that the flexural rigidity if flanges plays a very important part in the post buckled behaviour of such webs. For example, in Prague we obtained an increase in failure load of 130% by allowing for the flange strength, in Swansea Professor Rockey and I obtained an increase of 80%. That is why I think this factor is important, and I think that we should recommend the use of girders with rigid flanges (such as tubular ones) in order to improve the behaviour of thin webs. In fact this is one of the main advantages of the theory which Professor Rockey and I produced in Swansea and Cardiff. This theory makes it possible for the designer to allow for the effect of flange rigidity (and to design, for example, girders with tubular flanges); and, consequently, profit from the aforementioned very considerable increase in ultimate strength.

Any theory that does not take account of the influence of flange rigidity is bound to disregard the most important aspect of the behaviour of webs in shear.

PROF. L. BEEDLE.

You mean an increase of 30%?

DR. M. SKALOUD.

No, an increase in ultimate load of 130% by allowing for the beneficial effect of flange rigidity.

PROF. C. MASSONNET.

Would you say that you obtain a limit load of 230% of what Dr. Basler has calculated?

DR. M. SKALOUD.

230% of what we obtained when testing a girder with very flexible flanges.

PROF. L. BEEDLE.

Is that a practical proportion for the flanges?

DR. M. SKALOUD.

Yes.

PROF. C. MASSONNET.

Excuse me. In other words, would you say that the scatter between the girders with thinnest flanges and those with very thick flanges was 230%.

DR. M. SKALOUD.

Yes.

PROF. L. BEEDLE.

I would like to point out that Konrad used such terms in referring to the strength as ultimate load and ultimate strength, he did not say at one time 'collapse.' Now, this I think reflects in part his experience in the U.S.A. where we do not use terms that might give any kind of negative connotation to the designer. We would use the word 'collapse' to refer to the peak of a load deflection curve or, the point at which a column buckles. And I think that on many occasions today I have heard the word collapse used in the sense of plastic collapse which is really reaching a mechanism condition and sometimes corresponds to instability and sometimes does not. I think it worth thinking in terms of the next step of this type of work. We can talk at this meeting of γ^* and collapse but in the United States, you will not find these terms used. I have mentioned this as an aside for discussion but so many times we get into the habit of using symbols and glossary that mean something to all of use but which will lead to added confusion to the 50,000 engineers who will have to use it.

PROF. O. STEINHARDT.

This is of course a non-linear process; it is not a fracture and not an instability collapse. Therefore it is a limit load, not a collapse load and not a sudden instability.

MR. T. HOGLUND.

Can we tolerate the deflections which will take place if we use these ultimate load theories?

PROF. K.C. ROCKEY.

The designer has always to ensure that no unduly large deflections occur under working conditions. Dr. Skaloud has already indicated that in many of the tests on girders with relatively flexible flanges that we have conducted, we have obtained a relatively linear load/deflection relationship followed by a rather sharp failure. With stiffer flanges, you will obtain more of a plateau on the load/deflection curve and this is another reason why we should use fairly stiff flange assemblies. The designers are concerned about the deflections at the design load, not at the collapse load, and I think that these will be quite acceptable.

PROF. C. MASSONNET.

I also think that from this point of view the rigidity of flanges plays an important part.

DR. M. SKALOUD.

By using rigid flanges we get quite a slow collapse process; and, therefore, not such a dangerous type of failure as is encountered in the case of girders with flexible flanges when the failure is a sudden phenomenon.

PROF. O. STEINHARDT.

Only a remark: Supposing we have the limit load, and a safety factor for design. On the other hand we have the service load. Therefore we must tell Dr. Basler and others what real behaviour may be under the service load. One should not consider exclusively the limit load.

PROF. C. MASSONNET.

I suppose I could support you by saying that as in the concrete field, we have to design against two or three different limit states, one is the collapse state, the other may be fatigue

PROF. C. MASSONNET.

continued

danger, yielding of the web and so on.

PROF. O. STEINHARDT.

And you must know the real stresses in order to be able to design against fatigue.

PROF. C. MASSONNET.

Yes. I agree.