Free discussion

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Topic: **Ultimate Strength of Plate Girders Subjected to Shear**
Plate Girders without Intermediate Stiffeners

Résistance à la ruine des poutres à âme pleine soumises au cisaillement
Poutres à âme pleine sans raidisseurs intermédiaires

Tragfähigkeit schubbeanspruchter Blechträger
Blechträger ohne Zwischenaussteifungen

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PROF. L. BEEDLE Chairman's introductory remarks.

With respect to ultimate shear strength, which is the topic of the session in this first session of the Colloquium, the major efforts have largely been how to incorporate the strength of the flanges in the various flange/stiffener mechanisms that will be discussed this morning and also the refinements on the stress distribution within the web itself.

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

PROF. C. MASSONNET.

Dr. Fujii, would you tell us what is your final conclusion as a result of the comparison that you have undertaken of the various tests with the various design methods. Which is, in your opinion, the best model available for shear?

DR. FUJII.

That is a very difficult question, which I am unable to answer.

PROF. C. MASSONNET.

You will have to choose which method to introduce into the Japanese specifications. I am very interested in determining which is the best model.

DR. T. FUJII.

My assumption in the location in the plastic hinges does not agree with all of the test results, especially in the case of flexible flanges. However, from the point of the design the stiffer the flanges the better.

PROF. K.C. ROCKEY.

I wish, Dr. Fujii, first of all to compliment you on your presentation which has done much to clarify many points. I would agree with you that the simplest and possibly the best design procedure will be to ensure that the flanges are rigid enough to ensure that the hinge would develop in the centre of the flange or will not form at all. I think there are certain advantages in your procedure over that of others but the weakness is the one that you have mentioned, i.e. determining the hinge position with flexible flanges.

DR. M. SKALOUD.

I also think that it is important for a design theory to be able to allow for the flexibility of flanges; since this effect plays a very important role in the post buckled behaviour of webs in shear. It follows from the tests that were carried out by Professor Rockey and myself in Swansea and Cardiff and from other tests conducted in Prague, that the position of the inner hinge in the flange is a function of the flexural rigidity of the flange, Dr. Fujii's assumption being just a limiting case, which occurs for very heavy, massive flanges.

PROF. S. KOMATSU.

The heavy flange is not always suitable for the limit design of a shear panel, but the flange having rigidity given by formula (29) or (30) in my paper is the most suitable from the reasonable and economical point of view. Because the collapse of the flange will be induced at the time of the collapse of web plate. I would like to know any comment about the suitable flange rigidity to attain to the economical limit strength.

PROF. S. KOMATSU.
continued

I had not sufficient time to investigate Professor Rockey's recent theory, because I just received his report on my departure to Europe. However, my theory developed independently would explain sufficiently the test results of Dr. Skaloud.

I think, the position of plastic hinge of flange should be considered from this point of view. It was shown in the experiments by Professor Rockey and Skaloud that the ultimate load varied with the hinge position, and I think it seems to be not sufficient that the position of plastic hinge was not present on the actual point 'o' in Dr. Fujii's model. By allowing for the actual hinge position, one will be able to determine accurately the ultimate load for shear panel.

PROF. A. OSTAPENKO.

The paper presented describes a method of analysing longitudinally stiffened plate girders subjected to a combination of shear and bending. The method represents an extension of the method developed for transversely stiffened girders and is applicable to symmetrical, unsymmetrical, homogeneous and hybrid girders. The plan was to develop a general theoretical approach, however complex it might be, which would apply to a general case and give good correlation with test results. A simplified design procedure could then be developed on the basis of the numerical computer output. We developed the theory to our satisfaction, but the reduction to design formulas has been carried out so far only for transversely stiffened girders (Cardiff Conference), not for longitudinally stiffened.

According to the analytical model developed, the ultimate strength of a girder panel under shear consists of three contributions: the buckling strength of the web, the post-buckling strength of the web, and a contribution by the flanges which are assumed to form a panel type mechanism with plastic hinges at the ends of the panel as indicated in Figure 5a of our report.

The most obvious difference of this model from those proposed by Dr. Fujii, Professor Rockey and Skaloud, and Professor Komatsu is the formation of a panel mechanism by the flanges rather than of a beam type plastic mechanism with three

PROF. A. OSTAPENKO.
continued

hinges as they assume. Among themselves they differ in the method of locating the intermediate plastic hinge and in the pattern and intensity of the transverse pull by the web on the flanges. In our model, the pull was artificially taken into account by assuming the tension field intensity outside of the major tension field to be equal to one half of the major field intensity. The correction for the smaller or greater rigidity of the flanges was, in fact, made by the panel mechanism contribution (frame action). The extension of this shear model to the combined loading case for longitudinally stiffened girders is described in the report which was distributed. The model gave acceptably good correlation with the test results which were available to us. At this point I would like to ask Dr. Fujii how he analysed unsymmetrical girders using his method. It appears that his formulation assumes that flanges be equal to each other, yet his report lists comparisons with some unsymmetrical plate girders. The same question I would also like to ask Professor Komatsu.

PROF. S. KOMATSU.

I would use the weaker flange in the computation. If dealing with an unsymmetrical panel with both shear and bending like your case, I think it is better to treat in computation the weaker flange under consideration of axial compressive and tensile stress in upper and lower flange respectively.

PROF. A. OSTAPENKO.

However, when the larger flange is under compression, which one should be used?

PROF. S. KOMATSU.

The compressive flange in your specimen I think, because it seems to be weaker than the tensile flange by virtue of the effect of girder bending.

PROF. A. OSTAPENKO.

Well, since in case of pure shear both flanges are under zero axial load at mid-panel, your selection will be then according to the flange size. Now, looking over your shoulder

PROF. A. OSTAPENKO.
continued

at your report for the first time, I am not sure whether you applied your method to longitudinally stiffened girders. It appears, though, that you ended up with relatively simple design formulas, and this is an advantage over our method which still requires the use of a computer when there is a longitudinal stiffener.

PROF. S. KOMATSU.

When we wanted to get the optimum design of a flange, we were much concerned by the internal force transmitted to the flange by web loading. We have it acting on the flange in a limited state.

PROF. K.C. ROCKEY.

I think Professor Ostapenko has been very clear in his statement and this question of which flange you use also arises in the design of longitudinally stiffened girders. My philosophy here is that if you have a web plate with a longitudinal stiffener, let us say for example at mid-depth, then the lower half of the girder will act upon the upper half of the girder in the same manner as a rigid flange and similarly, the upper half of the girder will react to the action of the lower panel in the same manner as a rigid flange. Thus I would assume a 'diagonal' membrane stress field as shown in figure 12 of my paper. Obviously in practice we would not expect a hinge to form in the longitudinal stiffener. As you will note from figure 12, due to the interaction which occurs, that is, a "panel" acting in the same manner as a rigid flange to an adjacent panel, one does obtain an increase in strength as indicated by the hatched areas. The stress system is not necessarily symmetrical in a panel, and I think this is a region of a study that needs further investigation.

PROF. A. OSTAPENKO.

In other words, in the upper triangle there is a weakening effect due to the flexibility of the flange. But the right bottom triangle in the top sub-panel should be fully effective as in a complete tension field. The same also applies in a reversed manner to the bottom sub-panel. Thus, the tension in

PROF. A. OSTAPENKO.

continued

the two triangles adjoining the longitudinal stiffener will be extending into each other and combining into one.

PROF. K.C. ROCKEY.

Right.

PROF. A. OSTAPENKO.

However, when the stiffener is not at mid-height, the stiffener will give in more towards one sub-panel, and this probably should be looked into. In our model with the hinges assumed to develop at the ends of the stiffener this difficulty is by-passed, granted, in an artificial way. The only justifications for the model are that it maintains structural equilibrium, satisfies pseudo-compatibility, i.e. continuity of gross deformations between the sub-panels, and gives good agreement with ultimate loads from tests. It seems each model has some advantages and my feeling now is that the best thing would be to extend the good points of each theory and combine them into one approach. One of the seemingly logical and desirable items to include would be the assumption of the fixity of the web at its horizontal edges.

PROF. P. COOPER.

I would like to comment on the Rockey and Skaloud model. I am particularly attracted by the diagram of the various possible failures which I think is very helpful. When you have a web reinforced by a longitudinal stiffener with an internal panel I presume you now have in fact a different flange assembly, and therefore you will get a different hinge location and the same thing that would be true I assume in using your model with the unsymmetrical flanges where the compression flange might be boxed in to improve the buckling strength.

PROF. K.C. ROCKEY.

Thank you Professor Cooper. Yes we would propose that if one flange was very strong and the other is relatively weak then you would develop different hinge positions in each of the flanges and that there must therefore be an accompanying increase in strength due to the presence of the stronger flange.

PROF. C. MASSONNET.

Well, in 1962 I conducted some tests on a plate girder which had light gauge steel tubular flanges and I developed a linear buckling theory for such built-in plates. I tested in Liege a rather large girder, 18 metres long and 1.2 metre deep, with a web depth/thickness ratio of 500, and I was struck by the excellent performance of this girder. But, you know, in Civil Engineering, you only build what you can calculate and you build according to your calculations. Now, the Linear Theory did not give any distinct advantage for this type of girder over the usual type and for this reason, as it is more expensive than the usual type, it was never used and I insisted in vain in various lectures, on the supplement of ultimate strength given by this type of girder. But now that we have the theory of Professor Rockey and Dr. Skaloud, I wish to ask you the question: would you believe that we should advocate that type of girder because we can come very near to the plastic design and that there is high performance?

PROF. K.C. ROCKEY.

Yes - I believe that you should - with a tubular flange you obtain two advantages, one by avoiding any flexibility of the web along the longitudinal connection, with the web one can assume fixed end conditions in respect of the buckling so that the buckling strength will increase and this therefore reduces the tension field membrane action. Secondly, because of the increased flexural rigidity possessed by the tubular flange, a mid-span hinge would not develop and you would be able to develop a full diagonal tension field action and therefore you would approach the ultimate shear strength of the web.

PROF. C. MASSONNET.

Just a minute - you believe that your theory would apply to that case?

PROF. K.C. ROCKEY.

Yes - you would be able to use our ultimate load method to design plate girders having tubular flanges.

PROF. A. OSTAPENKO.

But even a flange like this would come to develop hinges at the ends because of a continuing racking deformation. As I remember, Dr. Fujii suggested in an earlier paper using various kinds of rigid flanges: tubular, delta, etc., and gave their plastic properties.

PROF. K.C. ROCKEY.

You would have a full tension membrane action and then the subsequent Vierendeel girder or frame mechanism.

PROF. P. DUBAS.

Pour les poutres mixtes, les membrures tubulaires n'offrent guère d'intérêt, la dalle offrant un encastrement gratuit. Pour les poutres de roulement lourdes, on adopte quelquefois une membrure en té, par exemple en prévoyant une bande supérieure d'âme plus épaisse. Ce problème mériterait d'être étudié.

MR. G.B. GODFREY.

Professor Dubas has said that in the case of composite construction there is of course no need to have a tubular flange because of the stiffness provided by the slab, or alternatively he says the system which he has sketched should also be investigated.

PROF. K.C. ROCKEY.

If I may, I would like to refer to page 11 of Professor Komatsu's very interesting paper - the figure shown there of girder a.1. This photograph is most interesting because it highlights two features. Firstly, one can clearly see that plastic hinges have developed in the top flange and secondly we should note the distortion of the central longitudinal stiffener with accompanying distortion of the vertical stiffeners. Now if we adopt the tubular construction to which Professor Massonnet has referred, then we would clearly reduce some of this distortion of the stiffeners and the flanges and I wonder whether Professor Komatsu feels that in order to benefit, in the future, from these tension field actions we need to determine more exact laws for the design of the vertical and horizontal stiffeners.

PROF. S. KOMATSU.

I design the longitudinal stiffeners of the girders to have a stiffness just 1.5 times the value provided by Dr. Skaloud's formula.

PROF. A. OSTAPENKO.

One and a half times $-\gamma^*$?

DR. M. SKALOUD.

If I understand Professor Komatsu well, he refers to the formula given in my paper published in the Structural Engineer in September, 1962. The formula gives the moment of inertia of stiffeners to be rigid in the whole post-buckled range. It is γ^* - value, resulting from the linear theory of web buckling, multiplied by 3 - 7, according to the position of the stiffener.

PROF. C. MASSONNET.

You do not multiply this value by 1.5?

PROF. S. KOMATSU.

In order that the specimen develops the sufficient diagonal tension field action, I designed the stiffener to remain straight.

PROF. L. BEEDLE.

I have a question to Professor Rockey - looking at this photograph to which you refer, a.1. - where are the plastic hinges located?

PROF. K.C. ROCKEY.

It is very difficult to be precise, but examining the large panel you will note three white spots close to the compression flange and I would suggest there is a hinge in the flange just to the left of the central white spot, quite close to the middle of that panel. In addition, there are accompanying hinges developing close to the transverse stiffeners.

PROF. L. BEEDLE.

I ask this question because in the studies of plastic hinges forming in steel frames, you know - we do not really know where the hinge is - in this mechanism as a matter of fact there is no plastic hinge it will inevitably be in a strain hardening region, so that being too precise about where the

PROF. L. BEEDLE.

continued

hinge is located - we may be fooling ourselves.

PROF. C. MASSONNET.

But we need hinge locations to make calculations.

PROF. S. KOMATSU.

I think we should design flange stiffness or flange strength under consideration of the web slenderness ratio because too strong strength of a flange is uneconomical. We should consider the joint design of webs and flanges.

PROF. L. BEEDLE.

I call upon Dr. Clark.

DR. P. CLARK.

There has been considerable work on aluminium webs in connection with aircraft construction. Professor Rockey has done some excellent research in this area. In the United States, Paul Kuhn at the National Advisory Committee for Aeronautics, many years ago, developed design methods for thin-web aluminium girders. We thought that two things were needed, first to simplify the design procedures that Kuhn had developed and secondly to take some account of flange flexibility. Here we were not considering plastic hinge formation in the flanges but rather the effect, which I believe Dr. Steinhardt described, of the elastic flexibility of a flange on the distribution of stresses in girders. We have found for the cases we were interested in that failure was generally by yielding of the web, by tearing of the web, by buckling or crippling of the stiffeners, or by buckling of the flange, either local, torsional, or lateral buckling. We wanted to find the effect of flange flexibility on distribution of the stresses in order to predict the loads that would cause failure by these various methods. We made the simple assumption that, if we have very flexible flanges, we would have a stress distribution similar to Basler's assumption; if we have very rigid flanges, we would have diagonal tension at 45°; and, if we had something in between, we would have a component of diagonal tension similar to Basler's distribution plus a component of 45° diagonal tension,

DR. P. CLARK.
continued
the ratio between these two components depending on flange flexibility. Adding up the different components in this simple model (which includes a component of pure shear) we arrived at some formulae for the various stresses in the girder.

PROF. K.C. ROCKEY.

Dr. Clark's comments on the mode of failure in aluminium girders is in agreement with my personal experience with aluminium girders of bolted construction. I have not tested any welded aluminium girders but in none of my tests on aluminium girders of riveted or bolted constructions have I encountered a plastic flange mechanism failure. Because of the lower buckling stress of the aluminium, one either gets crippling of the vertical or horizontal stiffeners or flange failure due to buckling of the outstanding legs or by the diagonal tension field initiating cracks at rivet holes. So I do not believe that in bolted or riveted construction in aluminium we ever encounter a beam type mechanism such as encountered in welded construction.

PROF. O. STEINHARDT.

We have tested both welded and bolted.

DR. P. CLARK.

We do not have experience with welded thin-web girders, and for this reason I was very interested in your tests, which resulted in something approaching a mechanism.

PROF. K.C. ROCKEY.

But I would consider in Professor Steinhardt's tests the flanges of the aluminium girders are in respect of the mechanism considerations, very rigid flanges. However, you get rather more deformation i.e. inward deflection because of the lower modulus of elasticity and therefore one obtains a different redistribution of the stresses in the web plate.

PROF. C. MASSONNET.

May I just ask Professor Steinhardt a question?

PROF. C. MASSONNET.

continued

It is in connection with his tests on welded girders. As far as I know, when welded, most aluminium alloys experience a very high decrease in strength, up to 30-40%, which aluminium alloy do you use - a zinc alloy? Otherwise one obtains a huge loss of plastic strength.

PROF. O. STEINHARDT.

We do not use a zinc alloy. The influence of the yield point is not so important in these tests.

PROF. C. MASSONNET.

Not for shear tests, but probably for bending tests it would be important.

PROF. A. OSTAPENKO.

Do I understand correctly that the principal cause of failure was assumed to be the yielding due to all of these non-linear effects?

PROF. O. STEINHARDT.

Yes. I use exactly the same procedure as Massonnet and Skaloud.

DR. M. SKALOUD.

It was the beginning of yielding.

PROF. A. OSTAPENKO.

What boundary conditions were assumed by Djubek in his non-linear analysis and were the horizontal edges considered as straight?

DR. M. SKALOUD.

Regarding the boundary conditions, I think that the web was, in most cases, attached to rigid boundary elements.

PROF. K.C. ROCKEY.

This research work by Dr. Djubek was very good but unfortunately it was not developed into a form ideally suited for a design procedure. I think it would be very valuable if that further step could be undertaken.
Do you agree?

PROF. O. STEINHARDT.

Yes, but to make an attempt to keep to the physical behaviour at first and then to develop simple theory.

PROF. K.C. ROCKEY.

If my recollections are correct, Dr. Djubek did allow for some elastic deformation of the flanges and also he did not allow the flange to move out of the plane. Dr. Djubek's contribution being that he showed the significant effect of this flange rigidity upon the stress distribution developed in the web. Is that correct, Dr. Skaloud?

DR. M. SKALOUD.

Yes.

PROF. L. BEEDLE.

I call on Professor Bergfelt.

PROF. A. BERGFELT.

I would like to comment on my tests. In my contribution, I have reported them in two parts. One part, and that part is very small, concerns the shear. I have shown that my tests on girders with no intermediate stiffeners have given results that are near to those which are predicted by the semi-empirical theory of S. Bergman which was published in 1948. For such girders, the influence on shear of the stiffness of the flange is not so great. More astonishing is perhaps that the results from tests on girders with stiffeners are also near to what is predicted by Bergman. As shown especially by K.C. Rockey and M. Skaloud, the results for such girders are very much dependent on the stiffness of the flanges, but Bergman has no correction for this. The reason for the coherency seems to be mainly incidental and may be explained from the fact that most of the tested girders had rather normal flange thickness to web thickness ratios.

PROF. C. MASSONNET.

I think that all of us are very interested in these girders, which are not transversely stiffened. This is because of the increase of the salaries of fabricators and since transverse stiffeners cannot be welded auotmatically they are extremely expensive. For this reason, there is strong pressure

PROF. C. MASSONNET.

continued

on me in Belgium, to produce rules for the design of this type of girder. We know that the Swedish have been quite active in this field and that they produced some years ago tentative specifications originating from the work of Professor Granholm. I would like to ask you, Professor Bergfelt; do you agree with this tentative specification or have you rules of your own? Which are the safest?

PROF. A. BERGFELT.

The Swedish specifications to which I have referred, they are only preliminary but I consider them to be very good.

PROF. L. BEEDLE.

Following up on Professor Massonnet's question; for how long has this type of design been permitted? How long have girders been designed for making use of this lower safety factor?

PROF. A. BERGFELT.

It is three of four years and they are now under reconsideration.

DR. M. SKALOUD.

I have two short comments. In the first one, I would like to come back to a point which was discussed earlier, i.e. to plate girders with flanges of high inertia. This type of girder is becoming popular also in Czechoslovakia. For example, such girders were used in the case of the new Danube Bridge in Bratislava. This type of girder was chosen to increase the post-buckled reserve of strength of webs. The second comment concerns the Czechoslovak Specifications in regard to the design of web plates. Our design concept has gone through three stages. The first one was linear, entirely based on the concept of critical load. The second stage, reflected in the current Czechoslovak Specifications, is partly linear and partially it is based on post-buckled reasoning. This means that in our current specifications is given a complete set of formulae for designing webs by means of the critical load concept. However, apart from that,

DR. M. SKALOUD.

continued

there is also a paragraph which enables (those designers which wish to do so) to design webs with due regard to their post-buckled behaviour. This paragraph was written by Dr. Djubek (who dealt with the design of webs) and myself (who established the formulae for the post-buckled design of stiffeners). At the moment a new (third) stage is starting in our country. In it we would like to base the design of webs on their ultimate load behaviour, using the theories established jointly by Professor Rockey and myself in Swansea and Cardiff.

PROF. A. OSTAPENKO.

I would like to make a comment on the work by Bergfelt and Höglund. In our theory, we also made a comparison with tests on girders with large aspect ratios of the order of 2.5 and up to 5.5. These were the tests by Nishino, Okumura, Carskaddan and Basler. All of these specimens were subjected to a concentrated load with a stiffener under it. For the aspect ratios greater than about 3, the post-buckling contribution was found to be negligibly small and the ultimate strength was thus computed as a sum of the buckling strength (web fixed at the flanges) and the frame action. Consideration of the variation of the moment in such long panels led to a better agreement between the theory and tests. Your tests, however, cannot be validly analysed using our approach since there is no provision for a distributed loading applied to the flange -- in this case development of the frame mechanism is very unlikely.

PROF. A. BERGFELT.

I just wanted to ask if you have stiffeners at the end.

PROF. A. OSTAPENKO.

All of these specimens had bearing stiffeners at the ends and under the concentrated loads. I only make the comment I do not know what would happen if one was using distributed loading.

PROF. L. BEEDLE.

Can we have a discussion on this point please.

MR. T. HOGLUND.

I agree with you because I found that my curve gives the same result as Basler's results for values of a/b of three or four.

PROF. A. OSTAPENKO.

Dr. Basler's original report did not include for the strength of the flanges, however, when the flanges are very heavy, the frame action of the flanges may be quite substantial. Some tests were conducted at Lehigh in 1936 or so on panels with aspect ratio of about 3 and the flanges contributed on the order of 20%.

PROF. P. COOPER.

We have had several theories on collapse models posed and discussed earlier today for the shear strength of transversely stiffened girders, and I think it would be of interest to ask each person to comment on how their theory accommodates or provides information for the design and proportioning of longitudinal stiffeners.

PROF. L. BEEDLE.

That is very good, why do we not start in the order the original presentations were made, then you might also comment on any modifications, position or opinion in the light of the discussion we had this morning.

DR. T. FUJII.

I have not researched in this field of study and am unable to add anything.

PROF. A. OSTAPENKO.

The axial force in the stiffener is obtained as the sum of the contributions from the beam action, the horizontal component of the fully or incompletely developed tension fields of the sub-panels and the axial force due to the moment which develops due to the panel (frame) action.

PROF. P. COOPER.

The question was how can you use this to proportion longitudinal stiffeners or to determine the longitudinal stiffener requirements.

PROF. A. OSTAPENKO.

The strength of the longitudinal stiffener is calculated as that of a column with a cross section consisting of the stiffener itself plus a portion of the web plate which is equal to the plate thickness multiplied by a certain number dependent on the yield stress. When the stiffener is one-sided and thus eccentric, it is treated as a beam-column.

PROF. K.C. ROCKEY.

I believe this is a problem that has not received sufficient attention and I have already indicated that I believe much more work needs to be done in this particular field. I am very conscious that the Lehigh people have made a very good attempt to produce a model and a design procedure for longitudinal stiffeners and I think they are to be complimented for this. I have no model other than that I have proposed for the case of pure bending.

PROF. L. BEEDLE.

Do you have any observations, Professor Komatsu?

PROF. S. KOMATSU.

I agree that this is a most important problem. At this time, I employ the values of the stiffener rigidity provided by the elastic buckling criterion multiplied by a factor. i.e. γ of the designed stiffener is some multiple of γ^*.

PROF. C. MASSONNET.

I want to be very clear about this, because it is a very important question - do you use Klöppel's books and charts and you multiply the theoretical value called gamma star, given by the elastic linear buckling theory, or do you use a model similar to that advocated for by the Lehigh school?

PROF. S. KOMATSU.

I employ a multiple of γ^*. I am working on this problem at the present time - since the positioning of a longitudinal stiffener depends upon the combination of the shear and the bending stresses. For example, for the case of pure shear, the best position for a single stiffener is at mid-depth, but with bending stresses it must be placed adjacent to the compression flange at the $\frac{1}{5}$th depth position.

DR. M. SKALOUD.

I think that I can assist in explaining the way Professor Komatsu designed his stiffeners, since I have been talking to him about this question, and he told me that he had used the formulae which are given in my paper published in the September issue of the Structural Engineer in 1962. If it is so, I can clear up the basis of these formulae. It is the γ^* value multiplied by Massonnet's coefficient (3-7). Incidentally, these are the rules used for the design of stiffeners in Czechoslovakia at the moment, if it is required that the stiffener shall remain straight in the whole post-buckled range. Professor Komatsu - he will correct me if I am wrong - told me that they had multiplied these values by 1.5 in order to make the stiffeners of his girders more rigid still.

PROF. C. MASSONNET.

I would like to make a small comment, which is that everybody here is convinced that the stiffeners must remain straight up the collapse. A slide presented by Dr. Skaloud reminded me that a long time ago, in 1952 I believe in Brussels, I presented a small lecture about buckling of webs. Mr. Shirley Smith was in the chair and he mentioned that he did not see why the stiffeners had to remain straight up the collapse. Of course, Dr. Skaloud and others have said that it could be that the optimum situation is obtained with flexible stiffeners. However, I am personally convinced that stiffeners which remain straight up the collapse correspond to more efficient designs than flexible stiffeners.

PROF. P. DUBAS.

En première hypothèse, la méthode du coefficient m peut être également appliquée aux raidisseurs flexibles ($\gamma<\gamma^*$); les raidisseurs doivent avoir m fois la valeur γ donnée par la relation $k = f(\gamma,....)$ de la théorie linéaire.

PROF. C. MASSONNET.

It is not exactly the answer to my question, which is: what is the present philosophy, should everybody design stiffeners for remaining straight up the collapse or not, what is your opinion?

PROF. P. DUBAS.

Cela dépend du problème. On n'a pas toujours besoin d'aller au γ^*. Si la tôle est suffisamment rigide avec un raidisseur plus petit que γ^*, on ne va pas au γ^*.

PROF. C. MASSONNET.

So that you chose your stiffener according to the safety factor that you want to obtain?

PROF. P. DUBAS.

Yes.

DR. M. SKALOUD.

I do not think that it is always necessary for a stiffener system consisting of web flanges and stiffeners. We should optimise the girders as a whole. But, unfortunately at the moment we have not enough evidence in this respect. That is why we usually prefer designing perfectly rigid stiffeners. Then, we can divide the web into individual panels etc. But I am not convinced that this solution is always the most economical one. Further evidence in this line is necessary.

PROF. P. COOPER.

The reason I made the request is that I am perfectly aware of the inadequacy of the simple rule that I had proposed in an earlier paper for proportioning longitudinal stiffeners in shear panels and yet I also think that there

PROF. P. COOPER.
continued

must be other requirements than just taking the required stiffener stiffness at elastic buckling and then multiplying it by some factor. I personally think that this needs more study and that perhaps some of the models which have been discussed today can be extended to provide longitudinal stiffener requirements. For the idealised case of pure shear, it is not clear to me where longitudinal stiffeners should be placed, since the bottom sub-panel (the one which is in tension) does not behave the same as the one which is in compression with regard to its tension field. I wonder if Professor Rockey would comment on this situation.

PROF. K.C. ROCKEY.

I believe that Professor Cooper is correct: in unsymmetrical girders one might prefer to locate the stiffener away from mid-depth, in order to reduce the membrane shear load acting on the weaker flange, because if you ensure that the stronger flange is supporting as large a panel as possible, then you will fully utilize the strength in the upper flange and therefore, with unsymmetrical girders one could envisage a stiffener being closer to the weaker flange in order to get a balanced strength in both panels. I agree with you that I believe that whilst the great deal of work which has been done, evident by all the studies that are on this table, Japanese, American, European etc. has largely sorted out the problems of transversely stiffened plate girders, more data is required in order to reach a more complete understanding of the behaviour of longitudinally reinforced girders.

PROF. A. OSTAPENKO.

For our test specimens, we successfully designed the longitudinal stiffeners according to Cooper's approach. It does make sense to treat the stiffener as a column - and as a beam-column when it is one-sided. We found that the greatest contribution of the longitudinal stiffener was for the case of combined loads, that is, not under pure bending nor pure shear. Do not ask me why - that is what the tests show and that is what the computer program according to our theory gives. I have no physical explanation, but, for example, in one test on an

PROF. A. OSTAPENKO.
continued

unsymmetrical girder the increase under combined loads was about 44% whereas under bending or shear alone only 10-20%; See Figure 12 of our report.

PROF. L. BEEDLE.

Professor Rockey.

PROF. K.C. ROCKEY.

I appreciate the point that Professor Ostapenko is making. When you have a panel loaded in shear and bending, the longitudinal stiffener has a dual function. When used on a girder subjected to bending, it adds a little to the section modulus of the girder but its main function is to increase the buckling resistance of the webplate. In addition, as shown by Owen, Skaloud and myself,both the rigidity and spacing of the longitudinal stiffener can influence the ultimate load carrying capacity of the girder by stabilizing the compression flange against inward buckling.

However, when a girder is subjected to shear and bending, the stiffener increases the shear buckling resistance of the individual panels and therefore reduces the transverse load that is applied to the flanges by the shear membrane field. This shear membrane transverse loading has a much more significant effect upon a flange whose plastic modulus is reduced by the axial compressive forces, and it is because of this that a longitudinal stiffener can be so effective in the combined loading case.

PROF. P. COOPER.

There is another point regarding longitudinal stiffeners which Dr. Clark raised, that is, using them without transverse stiffeners. I wonder how far you can go in terms of panel aspect ratio before you lose the effectiveness of the longitudinal stiffener by virtue of not having it anchored by the transverse stiffeners.

PROF. K.C. ROCKEY.

From the buckling aspect, I do not think it too important and we have all of the relevant data but for cases of post buckling action that is a more difficult and important problem.

PROF. A. OSTAPENKO.

No, I think it should have some effect because, when a web panel buckles under shear or a combination of loads, a series of alternating buckles will form, but a transverse, I mean, longitudinal stiffener would prevent this and force the formation of smaller buckles.

PROF. P. COOPER.

But if you use a very long panel the stiffener has to be very stiff in order for it to have some effect.

PROF. A. OSTAPENKO.

No, the longitudinal stiffener has to span only the full wave of the original buckles to force the formation of the smaller buckles, not from a transverse stiffener to another transverse stiffener.

PROF. K.C. ROCKEY.

In the case of central longitudinal stiffeners you increase the shear buckling stress some four times, that's the point I was making.

PROF. A. OSTAPENKO.

Yes, but what is significant is that the effect of the longitudinal stiffener is for the length approximately equal to the girder depth. In other words, even if the longitudinal stiffener was very long for a large aspect ratio, it functions about the same as if it were spanning between transverse stiffeners spaced a little wider than the girder depth. This is also the reason why even a slender flange acts as providing fixed support to a web - it has to provide torsional restraint only between full waves of the buckles - not for its full length between transverse stiffeners.

PROF. L. BEEDLE.

Are there any further observations about the mechanism of the models for the shear panels as distinct from the longitudinal stiffened panels?

PROF. P. CLARK.

It appears that a number of different models give good agreement with all the available testing data. Is this telling us that the problem is not too sensitive to which model you use?

PROF. L. BEEDLE.

Use the easiest!

DR. K. BASLER.

Well, I would like to ask Professor Ostapenko, I have read his report and to me it seems that he is violating the lower bound theorem of plastic analysis since the web materials will be used over the yield twice.

PROF. A. OSTAPENKO.

Are you referring to my shear report?

DR. K. BASLER.

Yes.

PROF. A. OSTAPENKO.

Unfortunately, this report could not be distributed to all of you, but we hope to have it published. The problem referred to is that the web plate in the corner between the flange and the transverse stiffener is utilised in two cases of plastification: in the tension field and in the plastic hinge of the flange. And this is in violation of common sense. Yet, this simple analytical model gives good results. I would like to give in this connection a free quote from a book by Pearson -- "If you have a plausible explanation which correlates with facts, (in our case experiments) why look for complications?"

PROF. K.C. ROCKEY.

If we can bring in here a small point of Dr. Skaloud you might see that neither I nor Dr. Skaloud have ever suggested how you should never calculate the frame action. There is a reason for this.

PROF. A. OSTAPENKO.

Well, let me continue. In other words, we have a great simplification of what is really happening. In fact, very few tests indicate development of such a frame mechanism. In most cases, a plastic hinge develops in the flange at the left end and somewhere in the middle, but very seldom at the right end, although this is assumed in some models based on the development of a beam plastic mechanism. It seems that a typical moment diagram in the flange goes from a negative plastic moment at the left end to a positive plastic moment somewhere in the middle

PROF. A. OSTAPENKO.

continued

and then to some unknown negative - or positive - moment at the right end. Thus, the result is neither a beam mechanism nor a panel mechanism, but a combination of the two. Since the intermediate positive hinge tends to travel to the right in the process of loading, the plastification is spread over a portion of the flange. As an approximation, we assumed that it gets to the right end. Others assume it to stay more or less in the middle, using one or the other criterion, and introduce a negative plastic hinge at the right end to make the problem definite and solvable. In all these formulations, considerable simplifications are made at the expense of locally violating the continuity and/or the plasticity condition. Well, one of such locations of inconsistency in our model is the web portion at the flange-stiffener junction. In our design recommendation which will be presented at the Cardiff Conference, we neglect the web participation in the plastic hinge of the flange (Ref. A)*.

PROF. K.C. ROCKEY.

If we take the beam mechanism shown in figure 5 of the joint report by Skaloud and myself and add to is a simple sway (Vierendeel) frame mechanism, then the angles of rotation occurring in the flanges at their junctions with the left hand stiffener will increase, whilst those occurring at the junction of the flanges and stiffeners at the right hand stiffener will decrease. Thus, the final mechanism which we observe in a girder after collapse is a combination of the beam mechanism and the frame mechanism. However, we have observed in a few tests a "frame" mechanism which did not correspond to the simple Vierendeel mechanism and it is for this reason that we have not specified the type of "frame" action which can occur. In any case where the "beam" type of mechanism occurs first, the contribution of this "frame" action to the ultimate load capacity is quite small and can be neglected. If, however, you have very strong flanges which are able to carry the diagonal membrane field without forming a beam mechanism, then the Vierendeel action will clearly become more significant.