IV

DISCUSSION PRÉPARÉE / VORBEREITETE DISKUSSION / PREPARED DISCUSSION

Factors influencing flexural Cracking of Precast Reinforced and Prestressed Concrete Beams in the Light of possible Non-uniformity in Manufacture

Influence des défauts d'exécution sur la fissuration par flexion des poutres de béton armé ou précontraint

Faktoren, die Biegerisse von vorgefertigten Stahl- und Spannbeton-Balken in Anbetracht der Ungleichmäßigkeit der Erzeugung beeinflussen

PAUL W. ABELES
D.Sc.
Research Fellow
University Southampton
London

I. Introduction

Flexural cracking in reinforced concrete beams cannot be avoided, when the tensile strain exceeds the extensibility of concrete, and this becomes a special problem with relatively high tensile strains, as it is the case with large steel stresses in reinforced concrete or by application of partial (i.e. limited) prestressing. There are definite factors which influence cracking, as has been realised from extensive research; but variations occur in similar members, since it is impossible to obtain complete uniformity of the surface conditions of the reinforcement, of the strength and compaction of the concrete around the steel (which affect the bond efficiency) as well as of the correct positioning of the steel. Thus cracking is also subject to probability considerations.

In prestressed concrete the development of visible cracks can be avoided at will if the design is based on Class I of the CEB-FIP classification (i.e. when only compressive stresses occur at the tensile face at the service load) or it may be limited to micro-cracks with Class II (i.e. when limited tensile stresses are permitted). However, with Class III visible cracks occur at service load as with reinforced concrete. With prestressed concrete, in addition to the variations possible with reinforced concrete the magnitude of the prestressing force may vary from the specified value with the consequence that visible cracks may occur even in beams Class I at service load. This obviously would happen only with bad workmanship and insufficient supervision and/or if the design assumption about shrinkage and creep losses do not agree with the actual conditions. Such discrepancies may occur if average values for rather humid conditions are considered when not applicable (e.g. at a desert or in a heated room, where shrinkage and creep are much greater). In the present paper, however, such variations of the prestressing force are excluded, it being assumed that a basic amount of supervision is ensured and wrong assessments at the design state do not occur.

Crack control is important to avoid corrosion (the danger of which is less a question of crack width than of satisfactory density of the concrete around the bonded steel and of a minimum cover) and also for aesthetic considerations. There is a great difference between bonded and non-bonded steel as possible with post-tensioning, when the tendons must be protected against corrosion.

The author had in Austria 1933-37 an opportunity of studying the effect of cracking on more than 200 tubular and rectangular test beams, reinforced with high strength steel bars of a yield point of 60 to 70 kp/mm^2 (85 to 100 ksi)*.The use of such high strength steel as reinforcement of centrifugally moulded (i.e. spun) concrete masts was possible and feasible, since it was permissible to base the design solely on ultimate load conditions for a factor of safety of three. Comprehensive static failure tests were necessary with very favourable results, discussed later. Also a number of rectangular beams, reinforced with this high strength steel, were tested. In all these investigations crack measurements were taken. Again in 1964-67 the author carried out extensive tests at the University of Southampton. Some of them related to a high strength three-wire strand of a proof stress of approx.similar magnitude to the yield point of the Austrian bars, tested 30 years earlier. This is discussed in III.

Other Southampton tests involved reinforced concrete beams, containing nontensioned prestressing steel, in order to study cracking in prestressed concrete after the prestress has become ineffective. This was based on a CIRIA grant and will be discussed in IV. Between 1944 and 1962, when associated with British Railways, the author had an opportunity of investigating cracking of prestressed concrete in connection with static, fatigue and sustained loading tests. Further research in that respect has been carried out at DUKE University, USA, since 1965 and also at the University of Kentucky in 1967, with which investigations the author was associated. The effect of fatigue and sustained loading is briefly discussed in V, whereas possible variations in manufacture are investigated in VI. A successful introduction of non-destructive, at-random tests was carried out at British Railways, Eastern Region 1949-62 to ascertain uniformity of

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*Note: In this paper "psi" and "ksi" mean "lbf/in^2" and "kip-force/in^2" respectively (1 kip = 1000 lb); "kp", mostly used in Europe, is for "kgf"; the new SI unit "N/mm^2", not introduced elsewhere, is shown only on a few illustrations.

workmanship in the production of precast prestressed members and this is described in VII.

Based on his observation at all this research, covering many hundreds tests the author summarises in VIII the essential factors, influencing cracking, mainly based on the Southampton tests (IV) and includes some recommendations for design detailing to limit the crack width.

II. General Notes on Cracking.

In a plain concrete beam, flexural micro-cracks develop long before the beam fails at the flexural concrete strength (modulus of rupture) which is only a nominal stress in a homogeneous section. Evans was the first to observe flexural micro-cracks at about 50 to 70 % of the flexural strength in plain concrete beams (Ref. 1). At DUKE University it was possible a few years ago to obtain photo-elastic pictures of such flexural micro-cracks in unreinforced concrete beams (Ref. 2). Previously, the author had called such micro-cracks "invisible" cracks, as e.g. illustrated in Ref. (3). As shown in Fig. 1 (taken from Ref. 4), the author has assumed that flexural micro-cracking corresponds to the direct tensile strength which is 50 to 70 % of the flexural strength and is independent of the stress at which the cracks become visible. This cracking stress depends mainly on the cross section and on the distribution of the steel near the tensile face, restraining the cracks, and also on the concrete strength. It approximates the stress at which the theoretical deflection lines of the homogeneous and cracked sections meet, as indicated in Fig. 1. If the steel is not well distributed, the cracking stress equals the modulus of rupture of the plain concrete. Micro-cracks develop when the limit of extensibility is reached and the maximum strain deviates from a straight line. Visible cracks develop after the deformation curve has already deviated from a straight line (see Fig. 1).

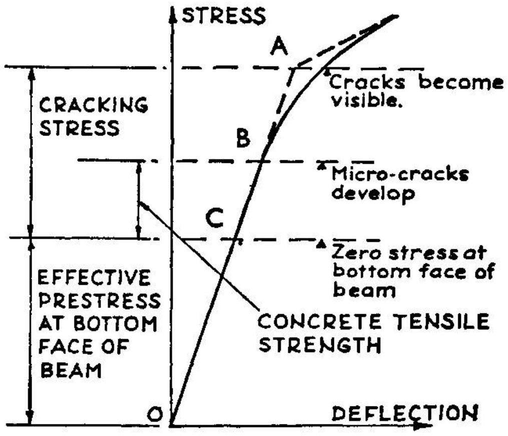


Fig. 1

For prestressed concrete beams, containing well distributed wires close to the tensile zone, the author found at many tests of British Railways a safe value of 1000 psi (70 kp/cm^2) for concrete of a cube strength of about 8000 psi, whereas with less satisfactory distribution it was 800 to 900 psi (56 to 63 kp/cm^2). With T-beams the cracking stress was as low as 630 psi (44 kp/cm^2). With rectangular and I-shaped beams the cracks are widest at the outer tensile face and gradually penetrate to the neutral axis, the steel restraining the width. With T-beams the cracks are restrained only close to the steel and become wider in the web, penetrating to the neutral axis near the slab, unless crack-restraining steel is provided at both sides of the web, as Roš suggested in Ref. (5) some time ago.

Good bond of the steel is of very great importance, greatly affecting the crack width. An under-reinforced concrete beam, containing large, smooth, wellanchored bars of unsatisfactory bond acts like a flat arch with a tie, few wide cracks developing similar to the conditions in beams with non-bonded tendons in which the cracks fork in the upper part. With well bonded and distributed, preferably deformed, bars many fine cracks develop similar to beams with pretensioned tendons. If a well distributed, bonded, non-tensioned reinforcement is provided also with non-bonded tendons a good crack distribution can be obtained, as shown in Ref. (6).

III. Cracking in Concrete Beams, reinforced with high-strength Steel.

The Austrian Reinforced Concrete Committee (Eisenbetonausschuss) in the early 1930's set up a sub-committee (of which the author was a member) to deal with cracking, and the Austrian reinforced concrete pioneer F. v. Emperger (Hon. ACI) investigated the expected crack width, based on the deformation in tensile tests, by means of mechanical strain gauges and showed that the maximum crack width at the position of the steel for the permissible mild steel stress of 12 kp/mm^2 (17 ksi) may be as much as 0.25 mm. (Ref. 7) This value was then considered as the limit which cannot be avoided in ordinary weak reinforced concrete. Higher steel stresses were only permitted if deformed bars and/or high strength concrete were used. At this time twin-twisted, work-hardened Isteg steel was used at a permissible stress of 18 kp/mm^2 (25.5 ksi). In the author's Austrian tests (Ref. 8, 9 & 10), however, high strength alloy bars (Siemens Martin steel) of a strength of 125 kp/mm^2 (177 ksi) was used. The results of the tests on spun concrete tubular beams are summarised in Fig. 2, showing for various percentages the calculated steel stress in a cracked section at a load when cracks became visible and at the working load (i.e. 1/3 of failure load) together with the corresponding widest cracks for two percentages. The appropriate working load stresses were 32 and 52 kp/mm^2, corresponding to failure stresses of 96 and 156 kp/mm^2, thus exceeding the yield point and with the small percentage even the strength. Thus full use was made of the strength of the steel. With small percentages cracks became visible rather late, but their width was immediately great, whereas with large percentages the cracks became visible at an earlier stage, but increased to a lesser extent. It was a special feature that the cracks completely closed on removal of the working load. This must be attributed to the excellent bond between the round steel bars and the high strength spun concrete which is illustrated in Fig. 3, showing the co-operation of the concrete tensile zone in spite of cracks.

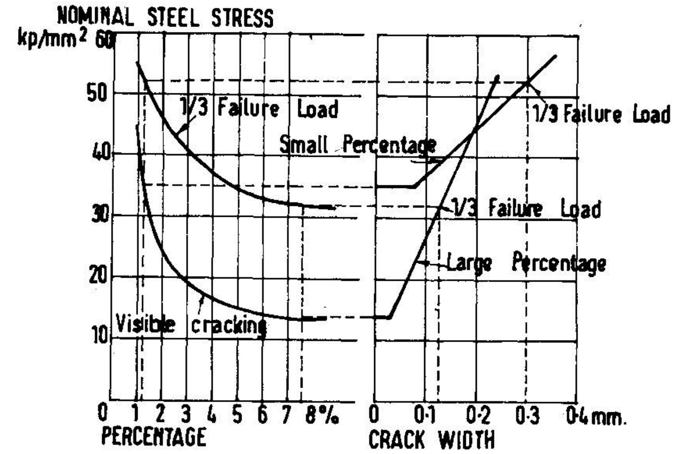


Fig. 2

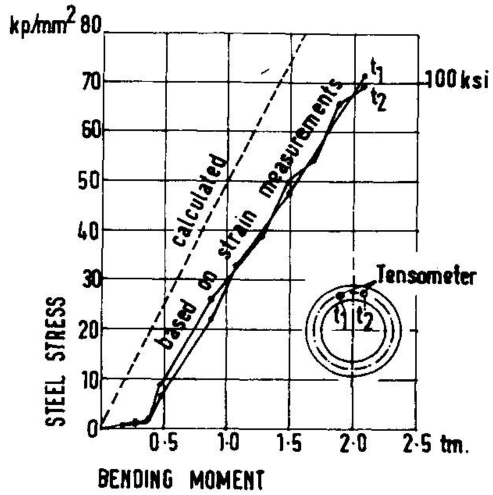


Fig. 3

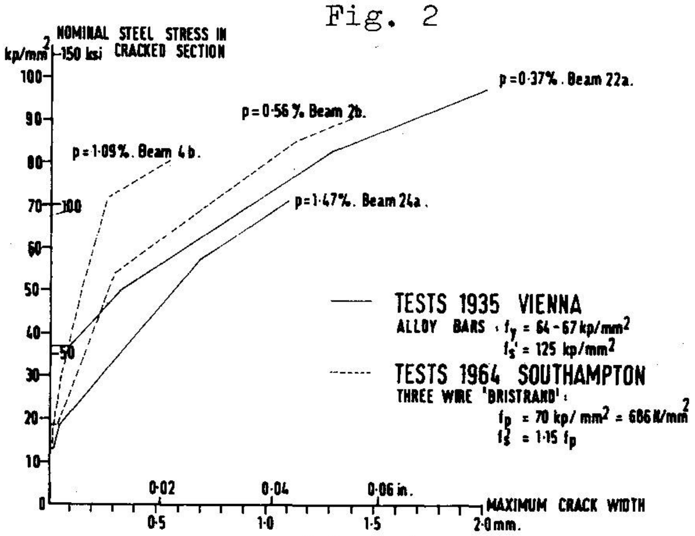


Fig. 4

In addition, about 30 rectangular beams of a size of 0.20 x 0.23 m and a span of 3.40 m were tested with two point loads. Strain gauge readings and crack measurements were made and similar results were obtained to that shown in Fig. 3. Fig. 4, taken from Ref. (10), illustrates the relationship between the theoretical steel stress in a cracked section and the maximum crack width for two of these beams of a reinforcement of 0.38 and 1.47 %, giving also the theoretical steel stress at failure. The stronger beam had 3 relatively large bars (18 mm dia.)and the bond was not so good; hence the crack width is greater with the stronger beam, contrary to the results with spun concrete. The concrete strength at most of these tests varied between 440 and 590 kp/cm^2 (6,200 to 8,350 psi), but with lower strength concrete of 145 kp/cm^2 (2,040 psi) the bond conditions were unsatisfactory except for the very small percentages. In Fig. 4 also two results of the Southampton tests 1964-67 are plotted, related to beams, containing three-wire strands which have a much better configuration than round bars and consequently also a better bond resistance, although the concrete strength was slightly less (Ref. 11). This steel has no distinct yield point, but a similar proof stress, however, with a lower strength (137 ksi (96 kp/mm2)). Although these beams have a greater reinforcing percentage, the theoretical steel stresses at the same crack width are much higher, which shows that the special configuration of the three-wire strand represents a useful improvement. Further tests were carried out on T-shaped beams, as reported in Ref. (11).

IV. The Southampton Tests 1965 (Relating to Pre-stressed Concrete).

The purpose of these preliminary tests was to obtain data about crack distribution and maximum crack width which would occur in prestressed concrete beams at increasing loading. This was accomplished by testing to failure two series of high strength concrete beams of different size, reinforced with nontensioned prestressing steel, ten types of beam being used in each series (see Fig. 5). These tests simulate the nominal concrete stress conditions at the tensile face of prestressed concrete beams of similar cross section, reinforcement and strength properties at loadings equal to, and exceeding those at which the effective pre-compression in the concrete tensile face has become zero (see Ref. 12 & 13).

The loading was carried out in three cycles. The first cycle limit was a load approximately half the expected failure load or when the maximum crack to 70 % of failure and the third to

SOUTHAMPTON TESTS 1965

	REINFORCEMENT	1	2	3	4	5	6	7	8	9	10
SERIES A	TYPE	Wire							Strand	Wire	
Cover : 12·7 mm. CROSS SECTION· NOS 9–10, NOS 1–8	DIAMETER (mm)	3·3	5		7				8	3·3	5
	NO.	2	2	4	1	2	3	4	2	3	3
	p %	0·19	0·47	0·95	0·45	0·92	1·36	1·83	0·92	0·44	1·13
SERIES B	TYPE	Wire		Strand				Alloy Bars			Strand
Cover : 25·4 mm. CROSS SECTION· NO. 10, NOS.1–9	DIAMETER (mm)	5		8		12·7		12·7		25·4	8
	NO.	2	4	2	3	1	2	1	3	1	3
	p %	0·19	0·38	0·35	0·51	0·43	0·88	0·60	1·89	2·57	0·89

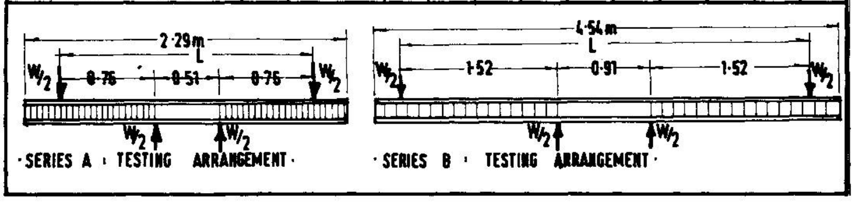


Fig. 5

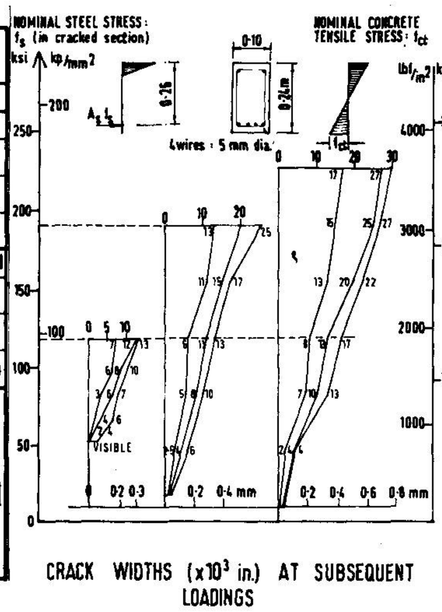


Fig. 6

failure. At each loading the central deflection was measured and, as soon as cracks became visible and measurable, the widest cracks between the loading points were measured at the tensile face and at the levels of the steel; moreover, the position was marked to which the cracks penetrated at each loading. Micro-cracking and visible cracking was observed by means of photo-elastic coating, as briefly reported in Ref. (14).

Fig. 6 illustrates an example of B series, containing 4 prestressing wires 5 mm dia., showing the measured maximum crack widths for various loadings at the three load cycles. In this figure also the nominal concrete tensile stress in a homogeneous section and the theoretical steel stress in a cracked section are plotted. In each of the three cracking diagrams, three lines are shown of which the largest crack width relates to the outer tensile face whereas the other two lines refer to the maximum crack widths at the level of the steel at the two sides of the beam. These crack widths are approximately equal at each side, if the cover is the same. However, in this specific case, the covers were different with consequent variations in maximum width. Generally, the maximum crack width at the upper range of the previous cycle was approximately the same at the subsequent loadings, but with lower loads the maximum crack width was usually greater at subsequent loadings. A comparison of the three crack width diagrams shows, however, that regularity of maximum crack width cannot be ensured, because new cracks may develop and consequently at a later loading the maximum crack width at a definite loading stage may be less than previously. For further particulars see Ref. (14), where also crack width measurements of another B-beam, containing 2 strands, are shown.

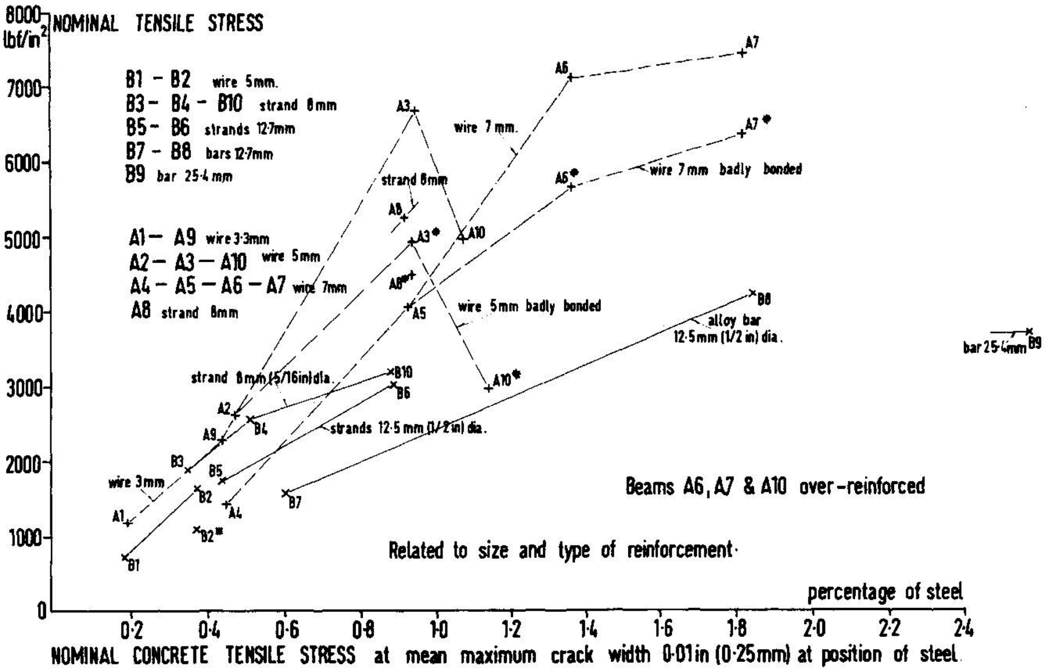


Fig. 7

The surface conditions of the steel were of great importance. Initially, completely smooth wires were used in a few beams, but much more favourable results were obtained when the tests were repeated with beams having slightly corroded wires. This is seen from Fig. 7 in which the mean results of Series A and B beams are plotted including 5 mean results for beams in which the steel had a smooth surface. Smooth surface is considered as a clean surface without any corrosion, but also without any lubricant. The latter possibility should be completely excluded, although it happened with two of the beams.

Generally, two beams for each type were tested and thus Fig. 7 relates to 50 tests. In this figure the nominal tensile stresses in a homogeneous section are plotted as ordinate, with the percentage as abscissa, for the mean values of similar beams, when the maximum crack width at the level of steel was 0.25 mm (0.01 in). The percentage is related to the effective depth of the steel and not to the double distance of the steel from the outer face, as often proposed, which would in this case, however, give similar results. The steel was not exactly positioned as planned and consequently differences occurred with regard to the effective-depth of steel and thus with regard to the percentage, as seen from the figure.

The results of the smaller beams Series A, having 12.5 mm ($\frac{1}{2}$ in) cover are much more favourable than those of the medium size beams Series B which have 25 mm (1 in) cover. In addition to the cover, the different size may have been of influence. In Fig. 7 the values of equal dia. of steel are compared. The lines joining the results B1-B2, B3-B4, B5-B6 and B7-B8 are almost parallel. Similar conditions apply to beams A2-A3 and A4-A5-A6. This indicates an improvement in conditions with increasing number of reinforcing members, as well as with better bond (size and shape of steel). There is an exception with A7, where a similar gain by increasing 3 to 4 wires does not occur. The cause seems to be less efficient compaction in view of the small space between the wires in the small beam Series A and hence satisfactory bond, which shows that there is a limit for increase in number. From the figure it is seen that also the number of reinforcing members independent of size and shape are of influence. If the results of beams containing 3 reinforcing members (B4-B8-B10) are joined, the resulting line is almost parallel to that for one reinforcing member (B5-B7-B9), which lines are not shown in the figure.

Based on the test results of the medium size beams Series B (thus ignoring the better results of Series A), safe values are plotted in Fig. 8, indicating simple relationships between the nominal concrete tensile stress f_{ta} and the maximum crack width at the tensile face respectively. This is illustrated for 3 different permissible crack widths specified by CEB-FIP (i.e. 0.30, 0.20 and 0.10 mm respectively) and in each case a difference is made between "wires and bars" and "strands" with its better bond. Fig. 8 relates to rectangular beams and a concrete strength of 8000 psi (560 kp/cm^2). A slight reduction migth be needed for lower concrete strength. Similar condition may apply to I-shaped beams, but differences are to be expected for T-beams. This would require further research. A lower limit of 0.3 % has been considered, because the cracks in beams of lower percentage became much wider soon after the cracks became visible similar to the spun concrete tests (illustrated in Fig. 2). The formulae for the relationships plotted in Fig. 8 are given there only in British units, but they can easily be converted. In Ref. (13) they are given in SI units (N/mm^2).

Fig. 9 shows the maximum crack width at a theoretical steel stress of 100 ksi (70 kp/mm2) in a cracked section of the individual beams which contained steel of approximately the same strength (i.e. wires and strands). In this figure the results of beams, containing only smooth wires, are distinguished from those, having a slight layer of corrosion (the latter being shaded). Extraordinary variations occurred between the individual beams. These differencies are much greater in beams with wires than with strands. In each of two cases with the greatest variation there was a lubricant on the steel at the position of the widest crack. These two cases were omitted from the mean values used in Fig. 7. It should be noted that the safe values on which the relationships in Fig. 8 are based relate to smooth reinforcing members, as it is difficult to ensure a uniform light layer of corrosion.

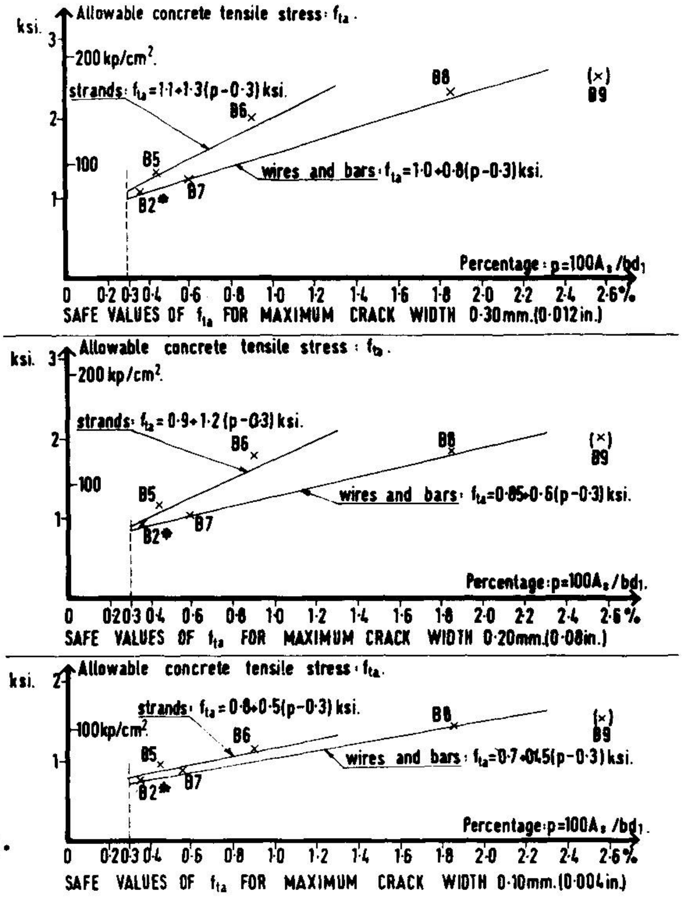


Fig. 8

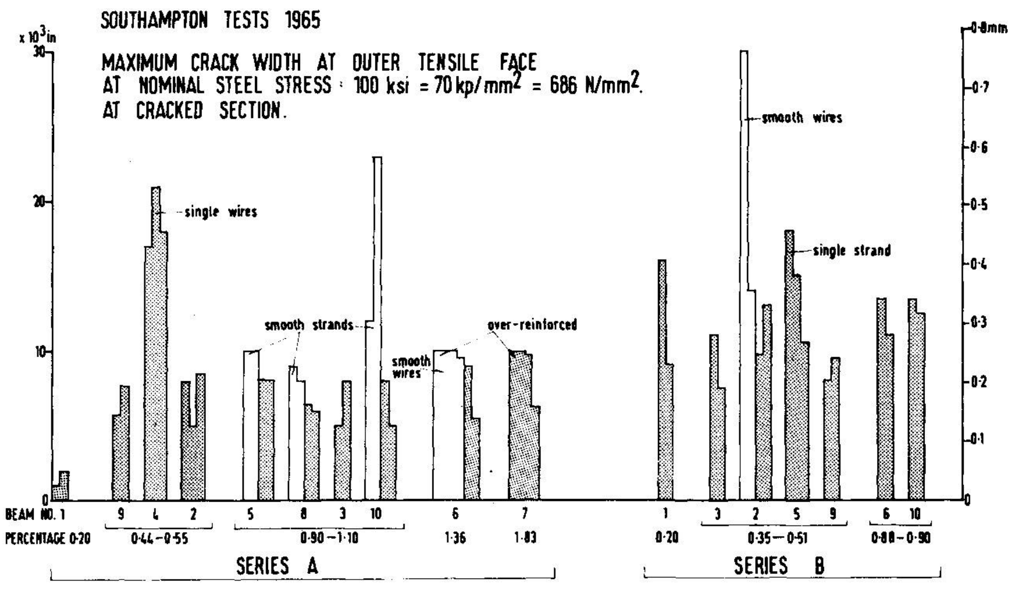


Fig. 9

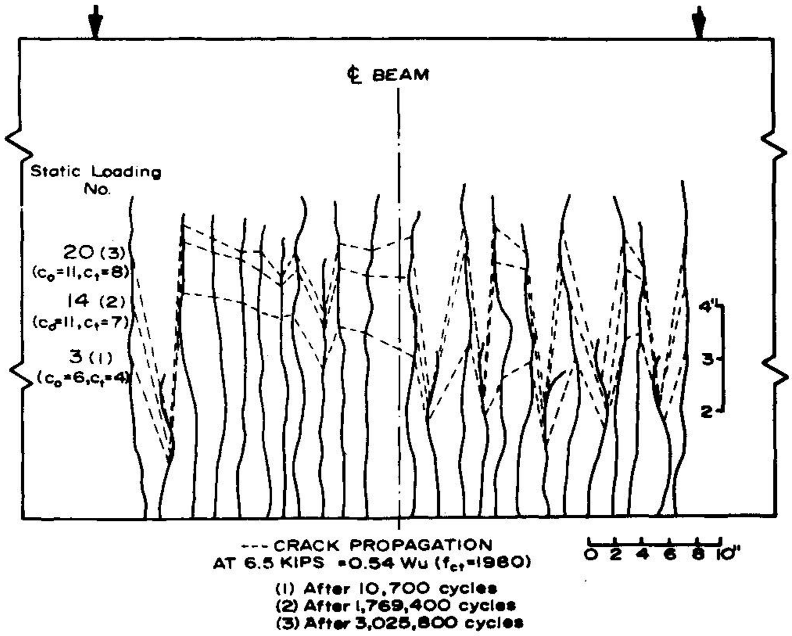
It is often propagated to measure all cracks, to determine the mean value of all widths and to compute the maximum crack width for each beam based on an assumed multiple of the standard deviation. In this case a very great number of fine cracks have to be measured which cannot be done with the same exactness as with wide cracks.

At the Southampton tests only the widest cracks at the tensile face and at the level of steel at both sides were measured at each loading of each loading cycle. It is usually easy to locate the widest cracks and to measure their widths. The same crack was not the widest at all loadings, because new cracks developed when the bond efficiency was not uniform along the beam. At the Southampton tests the propagations of the cracks were marked at each loading on the tensile face and the side faces of each beam. Before failure, these patterns of the cracks and corresponding loads were plotted so that developed plans of cracks are available, from which the spacing and the length of cracks at different loadings are seen. In the author's view, it is much more important to carry out many tests and to obtain the mean value from the widest cracks of each beam than to base this value on rather theoretical considerations, obtained from a great amount of less exact crack measurements, which take a considerable time to make.

V. Influence of Sustained and Fatigue Loading.

The effect of sustained and fatigue loading on cracking is rather complex. In Ref. (6) one example of a loading test, carried out by British Railways, is illustrated. A rectangular beam, containing well distributed pretensioned wires, was loaded after 920 days to half the static failure load when micro-cracks were observed. After approx. 150 days they became visible and increased gradually to 0.001 in (0.025 mm), the nominal concrete tensile stress being 880 psi (62 kp/mm^2). The max. crack increased to a width of 0.005 in. (0.125 mm) instantaneously, when the load was increased to 80 % of the failure load. This crack width doubled in very short time and increased to 3 times its size in 600 days. This was obviously an extremely high loading. However, in a companion beam at which the load was increased to only 63 % of the failure load, these cracks increased gradually from 0.002 in. (0.05 mm) also to 3 times this value in 720 days. In this case the nominal concrete tensile stress was 1970 psi (138 kp/mm^2), as described in Postscript Ref. (15). Other sustained loading tests with which the author was associated were carried out at DUKE University. For further particulars see Ref. (16).

Many fatigue tests were carried out at DUKE University, as described in Ref. (6) and (19). Fig. 10, taken from Ref. (17), illustrates propagation and crack width in a beam at different static loading cycles under a load of 54 % of failure. Fig. 10 has been distorted by using different scales for crack spacing and depth of propagation; c_0 means the crack width at the outer tensile face and c_t that at the level of steel. At a static load of 38 % of the failure load the well distributed cracks were very shallow (0.25 mm deep). This beam was instantaneously overloaded to maximum during the test, as described in Ref. (18), without ill effect. At present it is difficult to generalise the test results on fatigue.



Crack propagation in beam AL2* after various cycles of fatigue loading

Fig. 10

VI. Possible Variations in Manufacture.

The following variations may occur in the factors which influence cracking both in reinforced and prestressed concrete:

(1) positioning of steel; (2) its surface conditions and (3) strength and compaction of the concrete around the steel. Such variations occurred with the test beams of the Southampton research 1965. The effect of variations in positioning the steel greately depends on the size of beams; it is of particular influence with small beams. However, it is of great influence with regard to the cover independent of the size. If the reinforcing cage is correct, but wrongly placed, the cover may vary and wider cracks may occur, as e.g. indicated in Figs. 6 to 9. This applies not only to the sides but also to the tensile face. Similarly, variations in strength and compaction affect the crack width. In this respect the type of steel used is of great importance. If it is a round bar, the surface conditions are of much greater influence than with a bar having special configurations (deformed bar or strand). Since it is mostly impossible to ensure uniformity in surface conditions of the steel, smooth surface ought to be considered and in that respect a greater factor of safety, related to the stress corresponding to maximum crack width, is required for round bars than for deformed bars or strands.

Exactness of the prestressing force can be obtained only within definite limits. With good supervision and satisfactory measuring devices this should be limited to, say, $\pm$ 2½ %. This means that, for example, with an initial prestress of 3,000 psi (210 kp/mm^2) the difference may be 75 psi (5.3 kp/mm^2) which value decreases after the losses have taken place. This difference between specified and actual prestress is specially important with regard to Class II prestressed concrete to assess the required margin between cracking stress and permissible nominal tensile stress. The former may vary between 650 and 1000 psi (45 and 70 kp/mm^2) for rectangular and I-shaped sections dependent on the distribution of the steel in the tensile zone and the concrete strength, with lower values for T-shaped sections e.g. 550 to 650 psi (39 to 45 kp/mm^2).

VII. Non-destructive Testing of Prestressed Beams.

As already stated in I, very satisfactory acceptance tests were introduced by the author. The specified loading corresponded to 3/4 of the cracking stress, i.e. 750 psi (52.5 kp/mm^2) for beams with well distributed pretensioned tendons and 650 psi with grouted post-tensioned tendons. The effective prestress was based on the losses appropriate at the time of testing. (See Fig. 1).

These performance tests at the Chief Civil Eng. Dept. of British Railways, Eastern Region were regularly made on one member, selected at random from each pre-tensioning bed, and also on a certain proportion of members with post-tensioned tendons. Between 1949 and the end of 1962 about 1500 loading tests were satisfactorily made, in which no cracks became visible. During the first two years all beams passed the test and in a particular case of a job in 1958 all 88 roof beams, each about 60 ft. long, were successfully tested. (In this case the prestressing beds were short and only a single beam was made on each). There were cases at which beams did not pass the test and cracks developed; in all of these the cause of failure was established; for particulars see Ref. (15).

These tests were related to Class II in which cracks should not become visible at service load, but they could be modified to cover Classes I and III by specifying that the loading test should be continued until cracks become visible. This would not prevent the use of such tested specimens, because with Class I the cracks would remain closed under service load which is lower than the test load and with Class III the members are supposed to have visible cracks. The deflection diagram obtained at the loading test should be similar to that of Fig. 1. Thus it could be ascertained whether the prestressing force was correct, neither too large nor too low, bearing in mind the variations possible due to inexactness in the prestressing force and in the assumed losses of prestress.

VIII. Basic Factors influencing Crack Width. Recommended Design and Detailing.

The following factors affect flexural cracking: (1) <u>concrete properties:</u> strength and compaction; (2) <u>reinforcement:</u> percentage, shape, size, number and surface conditions; and (3) <u>geometrical dimensions:</u> shape of cross section, size of member, cover and spacing of steel.

Some of these factors are interconnected with each other such as percentage, size and number of steel reinforcement. "Good distribution" covers number and spacing of steel in the tensile zone. The influence of the bond efficiency on the crack width (depending on percentage, size, shape and surface conditions of the steel and on strength and compaction of the concrete around the steel, including shrinkage) should be obtained from flexural tests. Pull-out tests give only some comparative values and also tests to determine the bond resistance are not quite satisfactory. Recent tests to establish the required bond length (Ref. 21) indicated that different steel surfaces do not greatly affect bond slip, but they do affect the maximum crack width, as the Southampton tests have proved (see IV), although they did not have appreciable influence on the failure stress. As it is difficult to ensure a uniform layer of corrosion on the steel, it seems prudent to consider only smooth steel.

Many crack formulae have been proposed, based on specific research results for definite steel types and concrete strength. Usually they give a relationship between size (dia) and steel stress in a cracked section and in some cases also percentage, cover and spacing are considered. The Southampton tests have shown that the conditions are more complex. With prestressed concrete the nominal concrete tensile stress gives a good indication of the max. crack width. This value can be used for determining the required effective prestressing force (Ref. 20). With limit design of reinforced concrete, ultimate design applies to collapse load, and for the limit state of service ability the nominal concrete tensile stress might also be used for high strength reinforcement, in which case the classical design method for steel stresses could be entirely dispensed with.

Corrosion is less a function of crack width than of concrete density. The permissible crack widths of 0.3, 0.2 and 0.1 mm, specified by CEB-FIP as limits for different environments give only arbitrary corrosion protection. In fact, this depends in addition to density also on the composition of the concrete and on proper design & detailing. With dense spun concrete, 1 cm cover has proved satisfactory in the open air for permanently open cracks 0.3 mm wide (Ref. 22). In Ref. (23) the author described that spun concrete masts had well withstood the influence of heavy corrosive influences in spite of the small cover. Soretz showed that a cover of 1.5 cm is essential to avoid corrosion and suggests 5 classes with covers 2 cm to 4 cm (Ref. 24). Tests on well compacted prestressed concrete members by British Railways (see Postscript Ref. 15) have indicated that 1 in (25 mm) cover seems sufficient in the case of permanently open cracks of 0.01 in. (0.25 mm) width under very aggressive environmental conditions

The author is of the opinion that too wide cracks can be avoided by good design and detailing, as indicated in the following:

General design suggestions:

(a) base the required steel section on the collapse load with a percentage sufficient to limit maximum crack width at service load (e.g. Fig. 8);

(b) consider the crack width for normal service load, since that at rare service load is of no importance, as on its removal cracks close with good bond;

(c) with sustained or fatigue loading, the design should be based on small crack width, since it may increase to a multiple (say 2 to 3 times), unless the cracks are very narrow under static load (see V);

(d) additional prestressing (thus creating Class III) is an advantage as compared with ordinary reinforced concrete. This may be accomplished by the provision of additional, non-bonded, but corrosion-protected tendons.

General detailing considerations:

(i)provide the minimum permissible cover, ensuring sufficient corrosion protection and fire resistance, where necessary;

(ii) select steel of suitable shape (preferably deformed bars or strands);

(iii) select suitable size (dia.) of steel, ensuring good distribution;

(iv) provide relatively close spacing of steel members, but still ensuring satisfactory compaction;

(v) with deep T-beams provide crack-restraining steel at sides of the web.

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SUMMARY

Based on observations at many hundreds of high strength beam tests since 1933 (particularly Southampton research 1965), the factors influencing cracking are discussed. In addition to concrete cover, steel percentage, size and distribution (spacing), the bond efficiency (mainly dependent on shape, size and surface conditions of the steel and strength and compaction of the concrete around the steel) affects the maximum crack width. It can be limited by good design & detailing in spite of variations in manufacture (subject to probability). Efficient supervision with good workmanship is needed to minimise non-uniformity.

RESUME

A l'aide des observations faites lors de centaines d'essais sur des poutres à haute résistance depuis 1933 (surtout en 1965 à Southampton), on étudie les facteurs influençant la fissuration. Outre la couverture de béton, le pourcentage, la taille et la répartition de l'armature, c'est la qualité de l'adhérence des armatures (fonction de la forme, de la taille et de la nature de la surface de l'armature, ainsi que de la résistance et de la compacité du béton) qui influence la largeur des fissures. Celle-ci peut être limitée par une bonne conception de l'ouvrage et des détails de construction, malgré certaines variations dans la qualité de l'ouvrage. Une bonne surveillance lors de l'exécution permet de minimiser ces variations.

ZUSAMMENFASSUNG

Auf Grund von Beobachtungen an hunderten, hochfesten Balkenversuchen seit 1933 (besonders 1965 in Southampton), werden die für die Rissbildung wesentlichen Faktoren besprochen. Ausser Betondeckung und Stahlprozentsatz, Querschnittsgrösse und Abstandsverteilung, wird die grösste Rissweite durch die Güte des Verbundes (abhängig von Querschnittsform, Querschnittsgrösse und Oberflächenbeschaffenheit des Stahles sowie Festigkeit und Verdichtung des Betons) beeinflusst. Ihre Weite kann bei guter Detailausbildung trotz Abweichungen in der Herstellung (abhängig von der Wahrscheinlichkeit) auf ein bestimmtes Mass verringert werden. Ungleichförmigkeiten werden bei guter Ueberwachung und Ausführung abgemindert.

IV

On the Scatter in Yield Strength and Residual Stresses in Steel Members

Sur la dispersion de la limite d'élasticité et des tensions résiduelles dans les profiles d'acier

Über die Streuung der Streckgrenze und der Eigenspannungen in Stahlprofilen

LAMBERT TALL
U.S.A.

GÖRAN A. ALPSTEN
Sweden

INTRODUCTION

In determining the structural behavior of steel members subjected to compressive loads, the algebraic difference between the actual yield strength of each longitudinal fiber of the cross section and the residual stress existing in those fibers is of the utmost significance. Thus, it is important to know the magnitude and distribution of these characteristics as well as their variation inside a member and between different members.

This discussion summarizes some results obtained in various phases of a continuing study of residual stresses and column strength of rolled and welded steel shapes carried out at Lehigh University during the past twenty years. The different aspects covered in the paper include: (1) statistical variation of yield strength as obtained in routine mill tests; (2) comparison between results for yield strength obtained by various testing techniques; (3) variation of longitudinal fiber yield strength over the cross section of plates and shapes; (4) influence of strain rate upon the yield strength obtained in tension specimen tests; (5) variation or scatter of residual stress distributions as measured in members of same size and manufacturing conditions; and (6) scatter of residual stress distributions measured at various sections along a particular member.

MECHANICAL PROPERTIES

A graphical representation of the typical stress-strain curve for structural carbon steel, and the definitions used, are given in Fig. 1. The region corresponding to small strains is of primary interest here, that is, the elastic region and the yield plateau. In particular, the various yield strength levels as defined in different ways will be discussed in some detail.

Mill test results

The tension specimen test normally applied as a routine acceptance test for structural steels in U.S. mills is based upon the upper yield strength level, or, where an upper yield does not exist, upon the stress corresponding to a particular strain offset. The testing is performed in accordance with the techniques specified by the ASTM Standards. [1] The speed of testing is specified for the range from half the nominal yield strength through yield strength not to exceed 1/16 inch per minute per inch of gage length or, alternatively, not to exceed a stress rate of 100 ksi per minute. [1]

Figure 2 summarizes the results of routine mill tests carried out on 3,124 specimens of low-carbon steel (ASTM A7). [2] The average yield strength is 39.4 ksi for the whole sample, as compared to the minimum specified value of 33 ksi. The distribution is skew since material that does not fulfill the specified strength of 33 ksi generally is detected in routine control tests and not included in the sample. The yield strength of all specimens varied from a low of 31.1 ksi up to a highest value of 56.6 ksi. The shape of the distribution curve in Fig. 2 is similar to those of frequency distribution curves obtained in other investigations of structural steel. [2, 3, 4]

While the upper yield strength is used in specifying the strength of the steel in the U.S., the upper yield strength is of small or no significance in determining the yield behavior of a structural steel member. To use a statistical term, the validity of the upper yield strength concept with reference to structural behavior is small. This is well-known for members that are strained in a non-homogeneous manner (for instance, bending), but the upper yield strength is insignificant also for most homogeneously strained structural members. This is because practical members contain residual stresses which cause non-homogeneous deformations for homogeneous external loading conditions. Thus, the upper yield point is normally found only in a coupon as used in a tension or compression specimen test, that is, in a test essentially on a fiber of the material. The principal difference between the load-deformation relationships of a member loaded in pure tension or compression, and stress-strain curves with or without an upper yield level, is shown in Fig. 3. While the upper yield strength for a fiber is assumed 10 per cent higher than the lower yield strength level, the two load-deformation curves for the cross section differ only by the order of one per cent.

In characterizing a steel and for material standards, the present mill testing procedures based upon the upper yield strength concept appears satisfactory, since in this connection the strength value is relative. For discussions of the safety of structures, however, the difference between various testing techniques, and the manner in which a reported yield strength value has been obtained, may be extremely important and must be considered in the determination of the real safety of structures.

Effect of strain rate on yield strength

A second factor which influences the results obtained in a tension test is the strain rate. Investigations have shown that even a "very slow" laboratory strain rate used in testing tension specimens (a strain rate of 1 microinch per inch per inch second) may raise the apparent yield strength level by as much as 5 per cent. [5]

The effect of strain rate on the yield strength level in a typical tension specimen test is shown in Fig. 4. [6] The results are given in terms of the dynamic yield strength corresponding to a certain strain rate divided by the static yield strength. Tests were made for three groups of structural steel: ASTM A36 (Fig. 4a), A441 (Fig. 4b), and A514 steel (Fig. 4c), with specified yield strengths of 36, 50, and 100 ksi, respectively. Generally, the greater the strain rate, the higher is the apparent (dynamic) yield strength. The curves of best fit to the data points in Fig. 4 were obtained by regression analysis. The boundary curves on each side of the central curves represent the 95 per cent confidence limits of σ_{yd}/σ_{ys}. The range of variation between these two limits decreases with increasing nominal strength; for instance, the variation at a strain rate of 1,000 µin/in-sec is 9.0 per cent for A36, 7.0 per cent for A441, and 2.5 per cent for A514 steel. For strain rates normally used in the plastic range of a routine mill test, that is, of the order of 1,000 µin/in-sec, the apparent (dynamic) yield strength may be as much as 15 per cent above the "static" value.

Obtaining the static yield strength

The variation in the apparent yield strength as obtained by various testing procedures, and influenced by the different interpretation of results (upper or lower yield strength) and varying testing rate has led to the suggestion of a "static yield strength" testing procedure. The procedure simply prescribes one or more "stops" of the testing machine in the plastic range, and the stress level is recorded at the resulting zero strain. The static value at a strain of 0.5 per cent is usually recorded as the "static yield strength". See Fig. 1. The duration of the "stop" normally is from 3 to 4 minutes; during this time, the stress decreases gradually to the static value. ("Stops" in the elastic range normally give no appreciable decrease in load, which indicates that the static level is not seriously affected by inevitable mechanical inaccuracies in the testing machine.) Precautions must be taken so that unloading does not occur during testing, in particular, for hydraulic machines.

This procedure gives results which are independent of testing machine strain rate, and the "human factor". In addition, the results are relatively insensitive to inaccuracies in the alignment of the test specimen and to the effect of residual stresses which may remain in the test specimen. The method is applicable to materials with a definite yield plateau, such as structural carbon steel, as well as materials with a gradual transition from the elastic to the plastic range. Experience at Lehigh University over several years of testing has shown that this test method gives consistent and reliable results, applicable to the true yielding behavior of statically loaded structures.

Comparison between various testing techniques

A comparison between the results obtained in mill tests and in "static" tests is given in Fig. 5. [7] The different tests were performed on material from the same sample, representing steel supplied by two different companies in the form of hot-rolled H-shapes of 24 various sizes ranging from a 6WF15.5 to a 14WF426. The mill tests and the simulated mill test made on specimens from the web give average yield strength values of 42.3 and 40.6 ksi, respectively. The "static" values are 33.5 ksi for the weighted average of flange and web specimens (weighted with respect to sectional area of flange and web to furnish a yield strength value representative of the cross section of the shape) and 33.9 ksi for the stub column tests (compression test of complete cross section, tested for a member sufficiently long to retain the original residual stresses in the central portion of the member, but short enough to prevent column buckling). [2] It is of interest to note that the average mill test value for yield strength is almost 25 per cent higher than the average static yield strength obtained on the full-size member (stub column). On the other hand, the average static yield strength obtained from tension tests on flange and web specimens is very close to that of the stub column tests. The results seem to indicate that the geometrical influence of the specimen size on the static yield strength is small; also, from these results there is no apparent difference between the static yield strength in tension and compression.

Variation of yield strength over cross section

It was noted in the discussion above that representative yield strength characteristics were obtained when averaging the results of specimens from flange and web of H-shapes according to their respective areas in the cross section. The longitudinal fiber strength differs quite substantially between the flange and web elements, both with respect to yield strength (static as well as dynamic) and ultimate strength. Figure 6 gives a summary derived from previous results for flange and web specimens cut from a sample of 34 H-shapes of various sizes. [7] The average static yield strength is 33.0 ksi for the flange and 34.8 ksi for the web material, that is, a difference of 5 per cent. The difference in individual shapes is as high as 30 per cent. Only in two out of the total 34 shapes was the recorded yield strength of the flange higher than that of the web of the same shape. The difference in strength may be attributed partly to the position of the flange and web components with respect to the cross section of the ingot and the heat in the rolling process, and to the cooling behavior of the rolled member. The thinner web normally will cool faster than the flanges, resulting in a finer grain size and a higher yield strength of the web material.

The strength varies also within the individual components of the cross section. Figure 7 gives the variation recorded for yield strength obtained for 20 specimens taken from various positions in the flange of an H-shape type HE 200 B of a steel related to St 37. [8] The recorded yield strength varies between 32.8 and 41.9 ksi within the flange, that is, a variation of almost 30 per cent. Although the thickness of the flange is only 0.59 in (15 mm), the variation across it is quite significant.

Somewhat similar results were obtained in an investigation of the yield strength of tension test specimens cut from two heavy plates of ASTM A36 steel. [9] The plates studied were of dimensions 16x2 in and 24x3 1/2 in. Specimens were cut from two positions across the width of the plates, and five or seven specimens (for the 16x2 and 24x3 1/2 plates, respectively) were taken across the thickness of the plate at each position. Results for static yield strength of these tests are summarized in Fig. 8. The recorded yield strength varies between 30.7 and 34.8 ksi, for the 16x2 in plate, and between 29.5 and 33.7 ksi for the 24x3 1/2 in plate. The highest values were obtained in surface specimens, the lowest in interior specimens. This fact is consistent with the cooling conditions in the rolling process. The average static yield strength is 32.5 and 31.0 ksi for the 16x2 and 24x3 1/2 plates, respectively. These values may be compared to the reported mill test values for yield strength, 48.0 and 43.0 ksi, which are 48 and 39 per cent higher, respectively, than the average static values. The behavior of surface and interior specimens differed also in a more important manner in that the surface specimens showed a marked yield plateau and onset of strain hardening, while all the interior specimens had a gradual transition from the elastic to the strain hardening range. [9]

In conclusion, several tests indicate that the yield strength may vary over the cross section of a structural member. The variation may be quite significant, also when compared to the total variation between several different structural members manufactured from separate heats and at different mills. However, the variation is not solely statistical in nature, but to a certain extent predictable from the manufacturing conditions. The variation is of considerable importance in determining the strength of structural members; in particular, the variation must be considered when a small number of representative specimens are to be taken from the cross section of a member.

RESIDUAL STRESSES

Residual stresses exist in all practical structural members. While it would be possible theoretically to remove residual stresses from a member, for instance, by stress relieving, this is normally not practical or economical.

The residual stresses will vary inside a particular member; they will be in equilibrium. In addition, the residual stress distribution will vary from member to member of the same geometry and the same manufacturing and fabrication conditions, as well as between members of different geometry and manufacture.

A summary of all residual stress measurements performed would show a tremendous variation. A statistical treatment of the complete data from all measurements, irrespective of causes, is possible and straight-forward, however, this approach would be rather ineffective since most of the variation in results may be attributed to factors which could be controlled in the design or fabrication process. Thus, the studies of residual stresses at Lehigh University have been, in general, deterministic rather than statistical in nature. The major effort has been devoted to a study of the effect of various factors affecting the magnitude and distribution of residual stresses. Additional investigations were carried out to study the variation or scatter of residual stress distributions as measured in different members of the same geometry and manufacturing conditions and also the scatter of residual stress distributions as measured at various sections of a particular member.

The most important factors in the formation of residual stresses are the manufacturing and fabrication processes used, and the size and geometry of the structural member. [10] Any type of thermal or mechanical procedure used in the manufacture and fabrication, will affect, in general, the final residual stress distribution in the structural member. Thus, a hot-rolled shape normally will show a residual stress distribution quite different from that encountered in a welded shape of similar size. Residual stresses measured in various types of structural members have been discussed extensively in several papers. [2, 5, 7 through 17]

The variation or scatter in the residual stress distribution as measured at various positions along different members of same geometry and manufacturing conditions is exemplified in Fig. 9 for a hot-rolled H-shape, [11] in Fig. 10 for a welded box-shape, [13] and in Fig. 11 for a welded H-shape fabricated from flame-cut plates of A572(50) steel (σ_y = 50 ksi). [17] Figure 12 summarizes in histograms, the deviation between individually measured results in the component plates of the welded box-shape 10□65, and the average for the two different component plates, 10x1/2 in and 9x1/2 in, of all ten sections investigated.

In conclusion, the experimental studies summarized in Figs. 9 through 12 indicate that the variation or scatter in the residual stress distribution along a particular member or between different members of the same geometry and manufacturing conditions is reasonably small, that is, as long as the factors influencing the formation of residual stresses are uniform. On the other hand, it is obvious that discontinuous manufacturing or fabrication conditions, such as intermittent welding or local cold-straightening will lead to a wider scatter in residual stress characteristics.

From the above, it follows that residual stresses in a particular member may be predicted from information obtained on a similar member of the same geometry, provided the manufacturing and fabrication conditions are the same. An idea of the possible variation due to uncontrolled factors could be obtained from the results given in Figs. 9 through 12. The prediction might be obtained also from a theoretical study of the thermal-mechanical history during the manufacture and fabrication. [15]

The magnitude of residual stresses will affect the structural behavior of columns, and the important variable is the ratio between the residual stress and the static yield strength. For columns of hot-rolled H-shapes, the residual stresses at the flange tips are of primary interest. [2] Figure 13 gives an example of results obtained for 26 rolled wide-flange H-shapes; the sample is approximately the same as that of Fig. 6.

CONCLUSIONS

Various aspects on the scatter of yield strength and residual stresses in structural steel members have been covered in the paper. From the discussion, it is concluded that:

1. There is a significant difference between the apparent yield strength as obtained by various testing procedures. The variation is due to different interpretations of results (upper or lower yield stress), testing rate, and size and location of test specimen relative to the full-size member (that is, small specimen from a particular location of a member, or a test on a full-size member). Results obtained by the routine ASTM acceptance test normally used as the mill test in the United States may be 30% higher than the lower yield strength obtained in a very slow ("static") test.

2. Tests have shown that there is a functional relationship between the apparent yield strength level and the strain rate; the greater the strain rate, the higher the yield strength. The increase in the yield strength above the "static" value may be as much as 15 per cent for strain rates normally used in practice.

3. Based upon the variations in results obtained in different test procedures and at various strain rates, a "static" testing procedure was suggested to furnish test results which are independent of testing machine and strain rate. The procedure simply prescribes one or more "stops" of the testing machine in the plastic range, and the stress level is recorded at the resulting zero strain ("static yield strength"). Experience over several years of testing has shown that this test method gives consistent and reliable results, applicable to the true yield behavior of statically loaded structures.

4. There is a difference in yield strength between the various elements of a rolled shape, the thinner web normally being stronger than the flanges (the difference may be 5 to 20 per cent). There is also a variation in yield strength over the cross section of the elements of a structural member, in particular, for thick component plates. The variation in yield strength over the thickness of two thick plates (2 and 3-1/2 inches thick) was measured to be 10-15 per cent for A36 steel (σ_y = 36 ksi). In addition, the appearance of the stress-strain relationship was somewhat different between specimens taken from the surface or from the interior of the plates. Generally, the interior specimens showed no marked plastic plateau but rather a gradual transition from the elastic to the strain-hardening range. The surface specimens, on the other hand, followed the usual behavior with a separated elastic range, a yield plateau, and a

strain-hardening range. This variation must be considered when a small number of representative tension specimens are to be taken from a structural member.

5. The scatter of residual stress distributions measured at various sections along the length of a particular member appears to be small, as long as the factors influencing the formation of residual stresses are uniform. Thus, thermal residual stresses in hot-rolled plates and shapes or residual stresses due to continuous welding are more or less constant along the member. Measured variations in such members are of the same order of magnitude as the accuracy of the measurements. On the other hand, it is obvious that discontinuous manufacturing or fabrication conditions, such as intermittent welding or local cold-straightening will lead to a wider scatter in residual stress characteristics.

6. The variation between residual stress distributions measured in various members of the same size and fabrication conditions is reasonably small. Repeated measurements on different members, both hot-rolled and welded, have resulted in consistent results, the deviation between results normally being less than 5 ksi. Thus, residual stresses in a particular member can be predicted from information obtained on a similar member, provided the manufacturing and fabrication conditions are the same.

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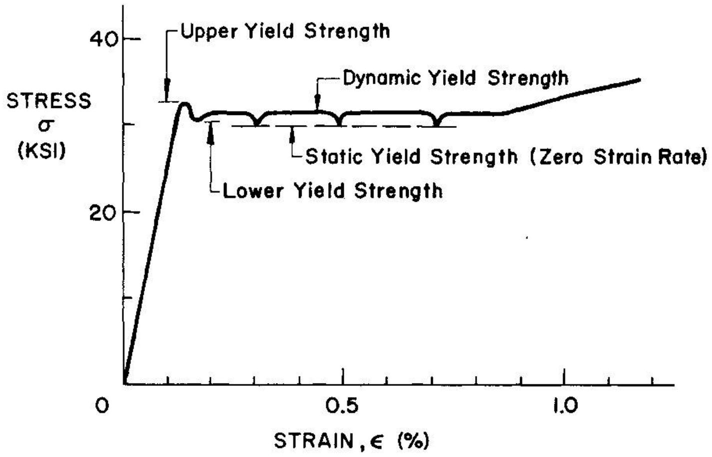
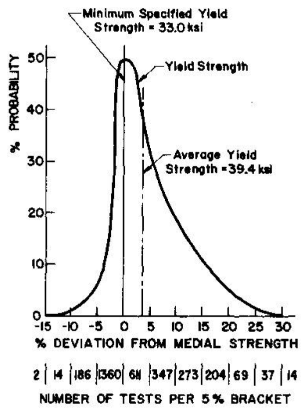


Fig. 1 Typical stress-strain curve of a structural carbon steel and definition of terms



No. of Mill Tests	3124
High	56.65 ksi
Low	31.09 ksi
Average	39.4 ksi
Standard Deviation	3.13 ksi
Probability Error	2.11 ksi 5.5 %
Coefficient of Variation	7.8 %

Fig. 2 Frequency distribution of yield strength obtained in 3124 mill tests

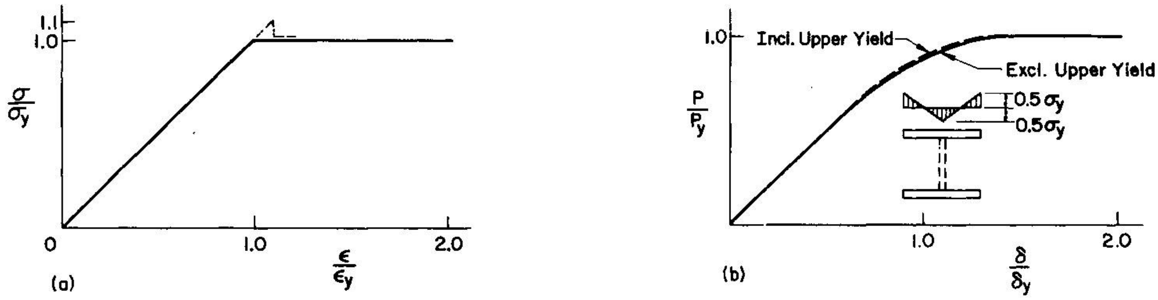
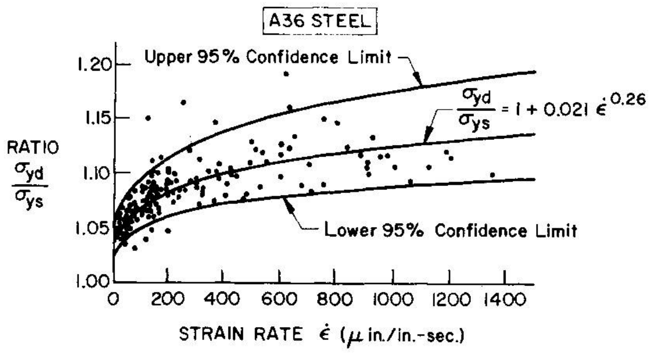


Fig. 3 Influence of an upper yield level upon the load-deformation behavior of a structural member containing residual stresses and loaded in pure tension (schematic)



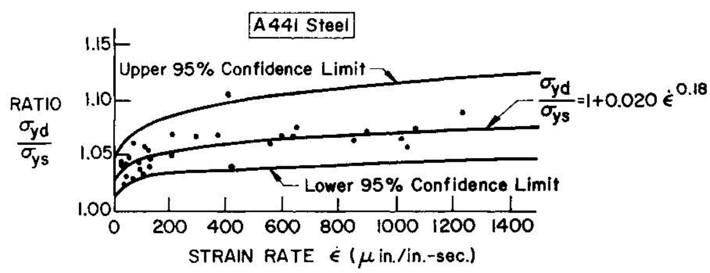
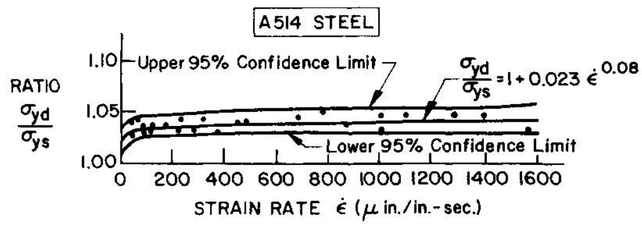
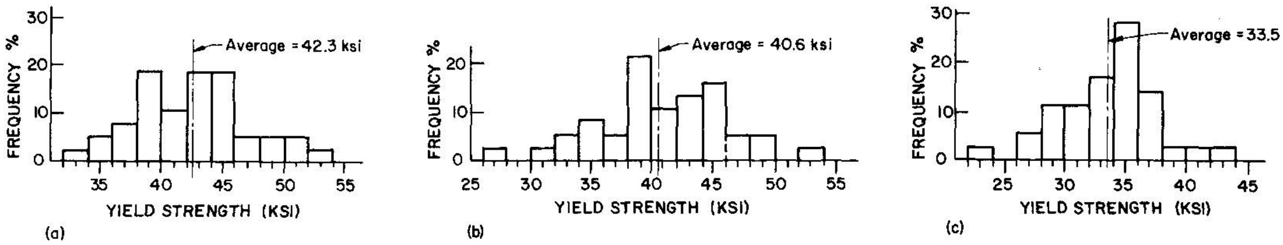


Fig. 4 Relationship between ratio of dynamic to static yield strength level and strain rate for ASTM A36, A441, and A514 steels





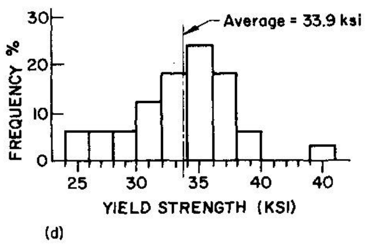
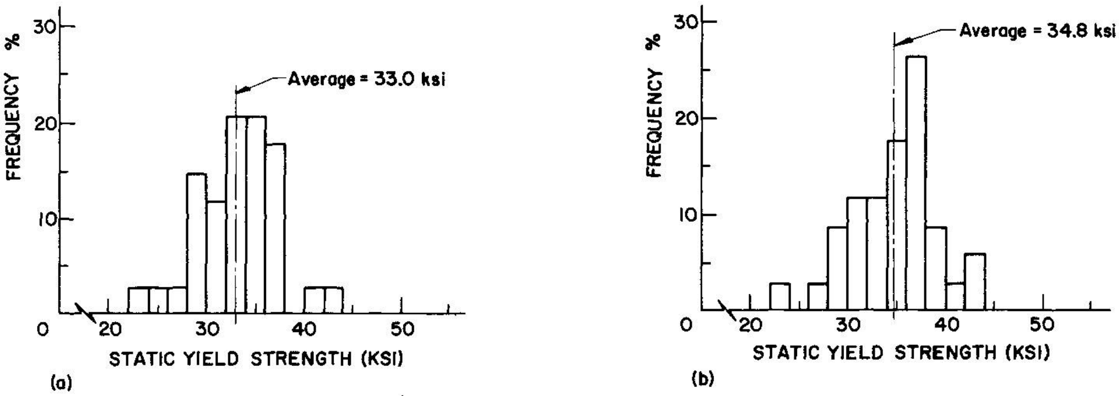
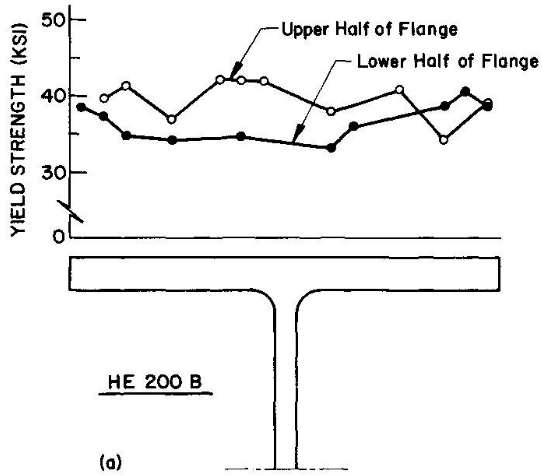


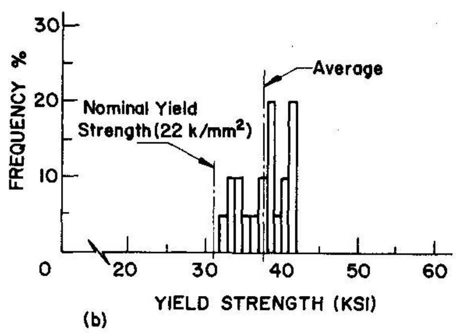
Fig. 5 Yield strength level as determined by various methods



(a) Specimens from Flange (b) Specimens from Web

Fig. 6 Static yield strength of specimens cut from the flange and web of 34 H-shapes

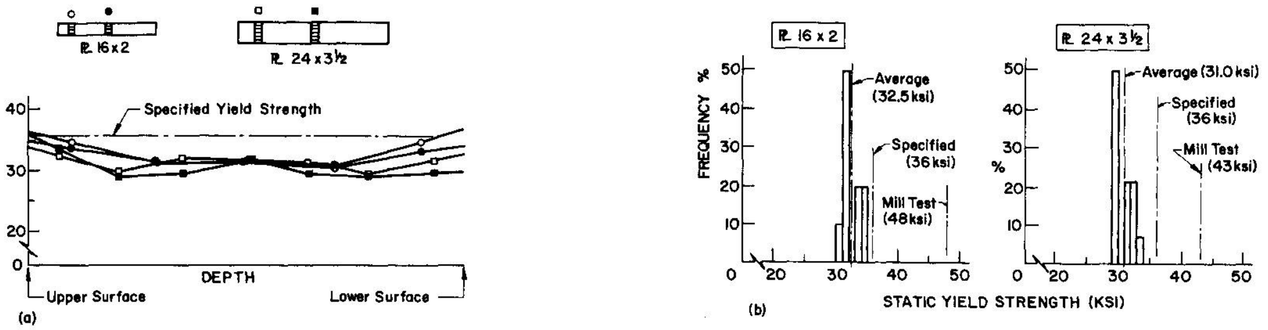




(b) Histogram

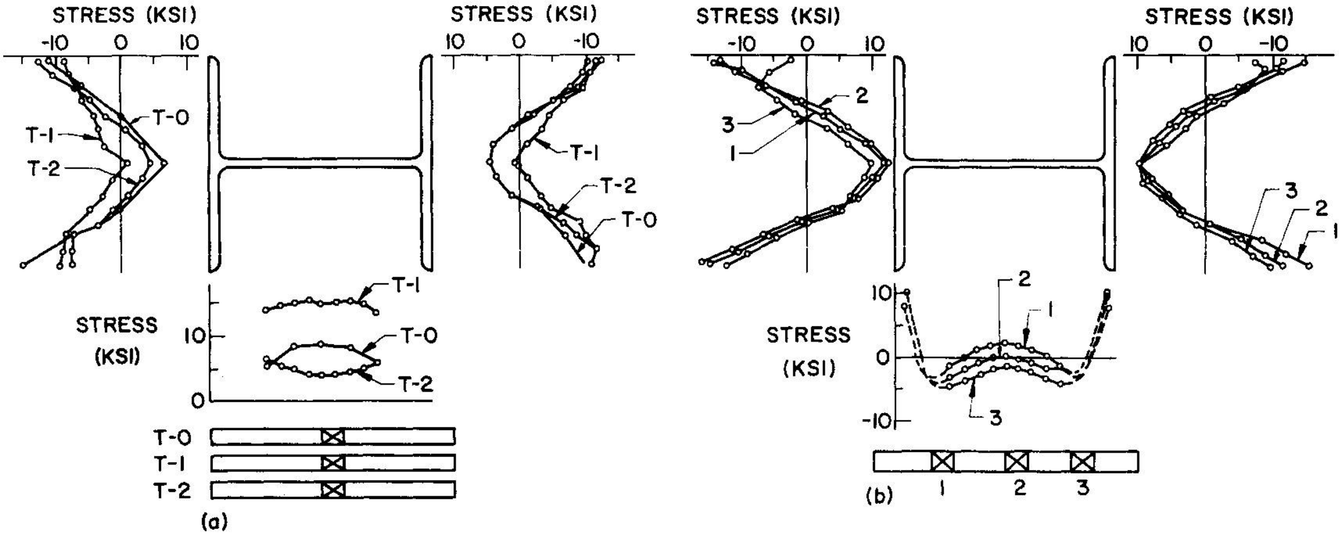
Fig. 7 Variation of yield strength for specimens from various positions in the flange of an HE 200 B shape

(a) Relationship between location and yield strength



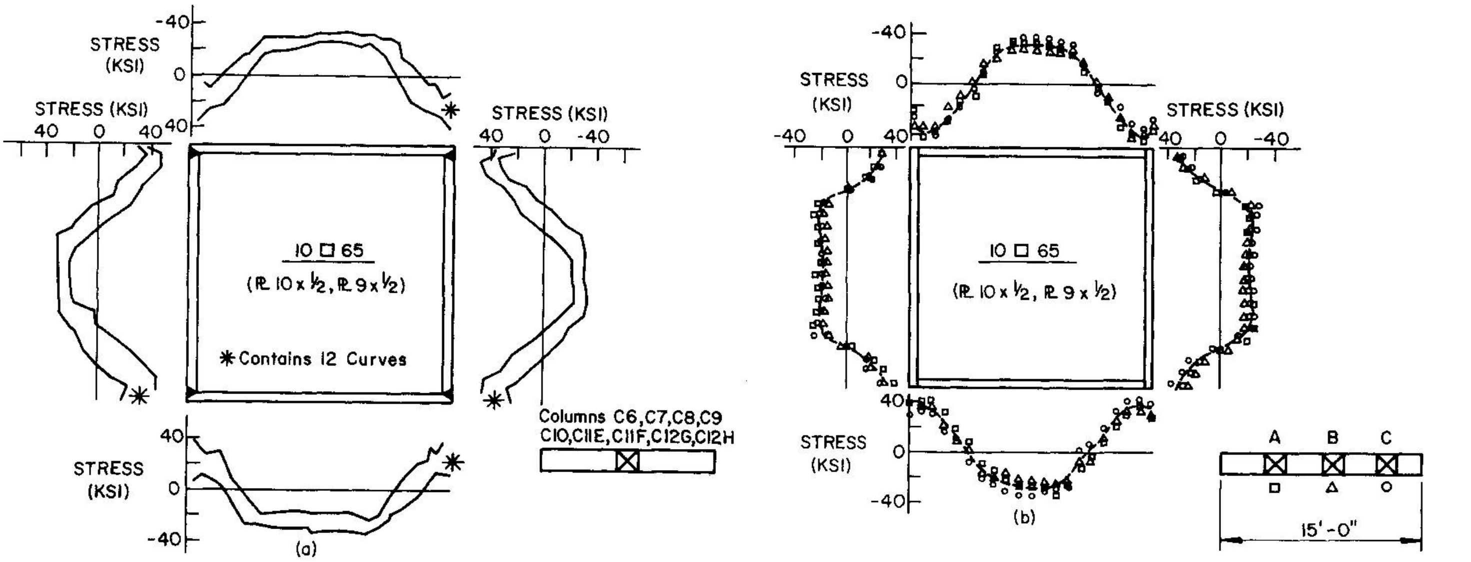
(a) Relationship between location and yield strength (b) Histograms

Fig. 8 Variation of static yield strength for specimens from various postitions of two thick steel plates



(a) Members T-0, T-1, and T-2 (b) Three sections of member T-6

Fig. 9 Residual stress distributions in four lengths of a hot-rolled H-shape, 8WF31



(a) Scatter between various fabricated lengths (b) Scatter within one fabricated length

Fig. 10 Residual stress distributions in several lengths of a welded box-shape, 10 □ 65

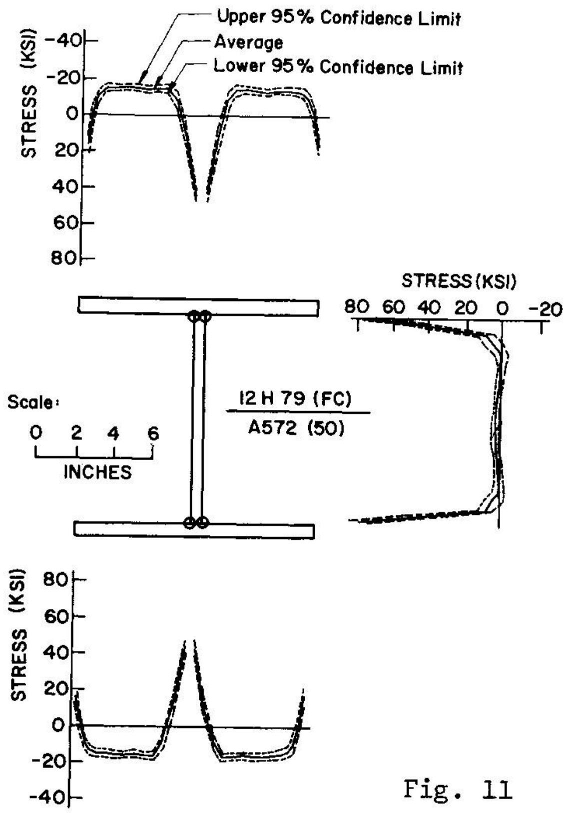


Fig. 11

Residual stress distributions in three fabricated lengths of a welded H-shape 12H79. ASTM A572 (50) steel with nominal yield strength of 50 ksi

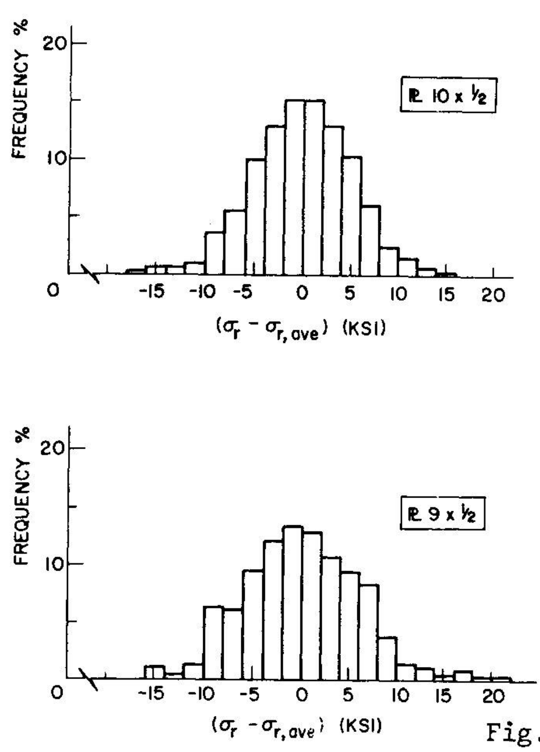


Fig. 12

Histograms of residual stress results obtained from 10 different fabricated lengths of a welded box-shape 10☐65 (same results as in Fig. 10)

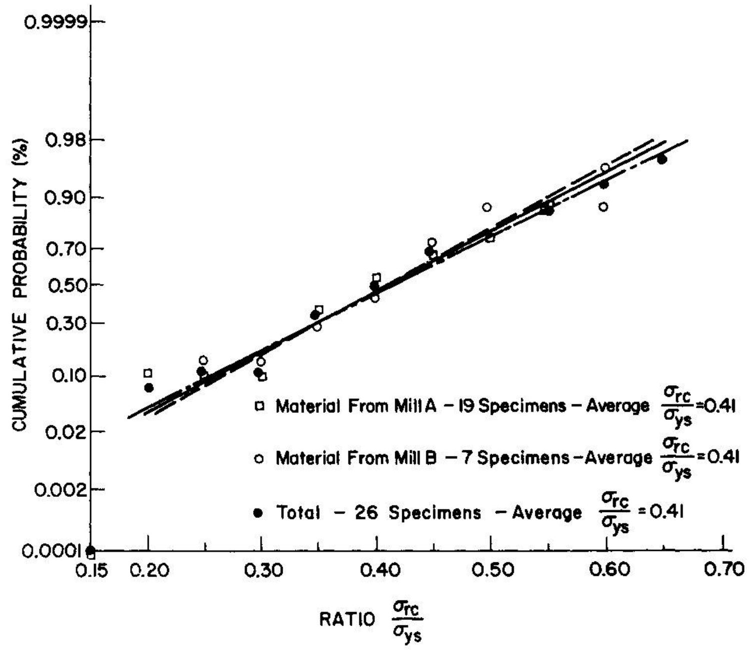


Fig. 13 Distribution of the ratio of maximum compressive residual stress to static yield strength for 26 hot-rolled wide-flange shapes

SUMMARY

The discussion summarizes some results obtained in a study of residual stresses and column strength of rolled and welded shapes. The different aspects covered include (1) statistical variation of yield strength in mill tests, (2) comparison of tension testing techniques, (3) variation of yield strength over the cross section of plates and shapes, (4) influence of strain rate upon yield strength, (5) variation or scatter of residual stress as measured in members of same size and manufacturing conditions, and (6) scatter of residual stress in various sections along a particular member.

RESUME

On présente ici les résultats obtenus lors d'une étude sur les contraintes résiduelles et la résistance des profilés laminés ou assemblés. Les différents aspects traités comprennent: (1) L'étude statistique de la variation de la limite d'élasticité donnée par les essais des laminoirs, (2) Comparaison des techniques des essais de traction, (3) Variation de la limite d'élasticité dans les sections droites des plaques et des profilés, (4) Influence de la vitesse de déformation sur la limite d'écoulement, (5) Variation ou dispersion des contraintes résiduelles pour des éléments de mêmes dimensions et de même provenance, et (6) Dispersion des contraintes résiduelles le long du même élément.

ZUSAMMENFASSUNG

Es wird über einige Untersuchungsergebnisse von Eigen- und Stützenspannungen an Walz- und geschweissten Profilen berichtet. Gesichtspunkte, die besondere Beachtung fanden, sind (1) die statistische Streuung der Streckgrenze bei Zugversuchen aus der laufenden Produktion , (2) der Vergleich zwischen Ergebnissen der verschiedenen Verfahren für Zugversuche, (3) die Streuung der Streckgrenze über den Querschnitt von Band- und Profilstahl, (4) der Einfluss der Dehnungsgeschwindigkeit auf die Streckgrenze, (5) die Streuung der Eigenspannungen bei Profilen gleicher Form und Herstellung und (6) die Streuung der Eigenspannungen in verschiedenen Schnitten einzelner Profile.

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IV

Variations in the Mechanical Properties of Structural Steels

La dispersion des caractéristiques mécaniques des aciers de construction

Die Streuungen der mechanischen Eigenschaften von Baustählen

M.J. BAKER
Research Assistant
Imperial College
London

1.0 INTRODUCTION

This report is limited to a discussion of some of the factors which influence the strength of structural steel members and in particular deals with the variations in the mechanical properties of steels and their effect on strength and safety.

1.1 Characteristic Strength As discussed by Rowe(1), semi-probabilistic methods are currently being introduced into British and European structural design recommendations which will require the designer to use 'characteristic values' for strengths and loads. The characteristic strength is defined as a load bearing capacity which will be exceeded by a prescribed percentage (taken as 95% in this report) of a population of similar elements or structures. With these methods, it is not necessary to know the exact distribution functions of these main variables, but the problem of assessing realistic characteristic values for different materials still remains. Where data are available, characteristic values can be calculated by suitable methods of statistical analysis, but the results may be misleading unless the origin of the data and the method of sampling are known.

For structural members which do not collapse as a result of buckling or instability, the yield strength of the steel is the predominant source of variability in the strength of the member. The variations in yield strength result from differences in chemical composition and strain/temperature history during rolling and subsequent handling. The reasons for the variability in the yield strength of steel plate and reinforcing bars have been discussed by Leclerc(2) and in general it can be concluded that the distribution function for the yield strength depends on:-

(i) the grade of steel produced, the type of product rolled and its approximate chemical composition

(ii) the thickness of the finished material

(iii) the characteristics of the steel mill (e.g. rolling sequence, temperature, etc.)

From the results given in Section 2.0 it can be concluded that changes in any of the above conditions result in systematic variations in yield strength and give rise to different distribution functions. If the above conditions are constant, the yield strength can then be considered to be a random variable, the variability being due to the combined effects of a considerable number of small random variations.

However, the statistical distribution of a series of tensile test results may differ from the distribution of the true average yield strength of the material they are intended to represent, because of systematic deviations of the recorded yield strength due to tests at high and differing strain rates(3), and because of the positions from which the test specimens have been cut.

2.0 VARIATIONS IN YIELD STRENGTH OF STRUCTURAL STEELS

The variability of several grades of structural steel has been investigated by the statistical analysis of results of tests obtained from different sources for the purpose of assessing characteristic strengths and in an attempt to compare the relative safety of structures constructed from different materials. In all cases (except Section 2.6) the data refer to tests on steel as rolled prior to any rejection of material of less than specified quality and not as delivered to the customer.

2.1 Mild Steel Plates to British Standard 15:1961* These data have been obtained from about 4000 tests relating to steel ordered by British Rail between 1961 and 1966, and can be considered to be a series of random samples selected from the five mills which supplied the steel. The test results below the guaranteed minimum yield strength originate from material subject to retest, but it is probable that some low values may have been omitted from the sample. This would have the effect of decreasing the observed variance. Some statistics are given in Figure 1 and from these results and further statistical analysis it can be shown that:

(i) there are significant differences in the mean and variance of the measured yield strength of steel supplied by different mills (compare mill 'V' and mill 'X')

(ii) these differences exist for all thickness ranges

(iii) the degree of control shown by mill 'V' is such that (a) changes in the target minimum yield strength for different thicknesses of plate is accurately reflected by the mean strength, (b) there is no significant difference between the variances for different thicknesses, and (c) the characteristic strength (based on a 95% probability of being exceeded, and an assumed normal distribution) is in all cases only a little above the guaranteed minimum value

(iv) there is a significant decrease in mean strength with increase in plate thickness

(v) for mill 'W', the characteristic strength is about 1 tonf/in^2 (15.5 N/mm^2) above the minimum yield strength for all thickness ranges (except one), and because of the consistently small variance, steel from this mill can be shown to give structures with the smallest probability of collapse

* These British Standard Specifications were in force at the time of production, but have now been superseded by B.S 4360:1968.

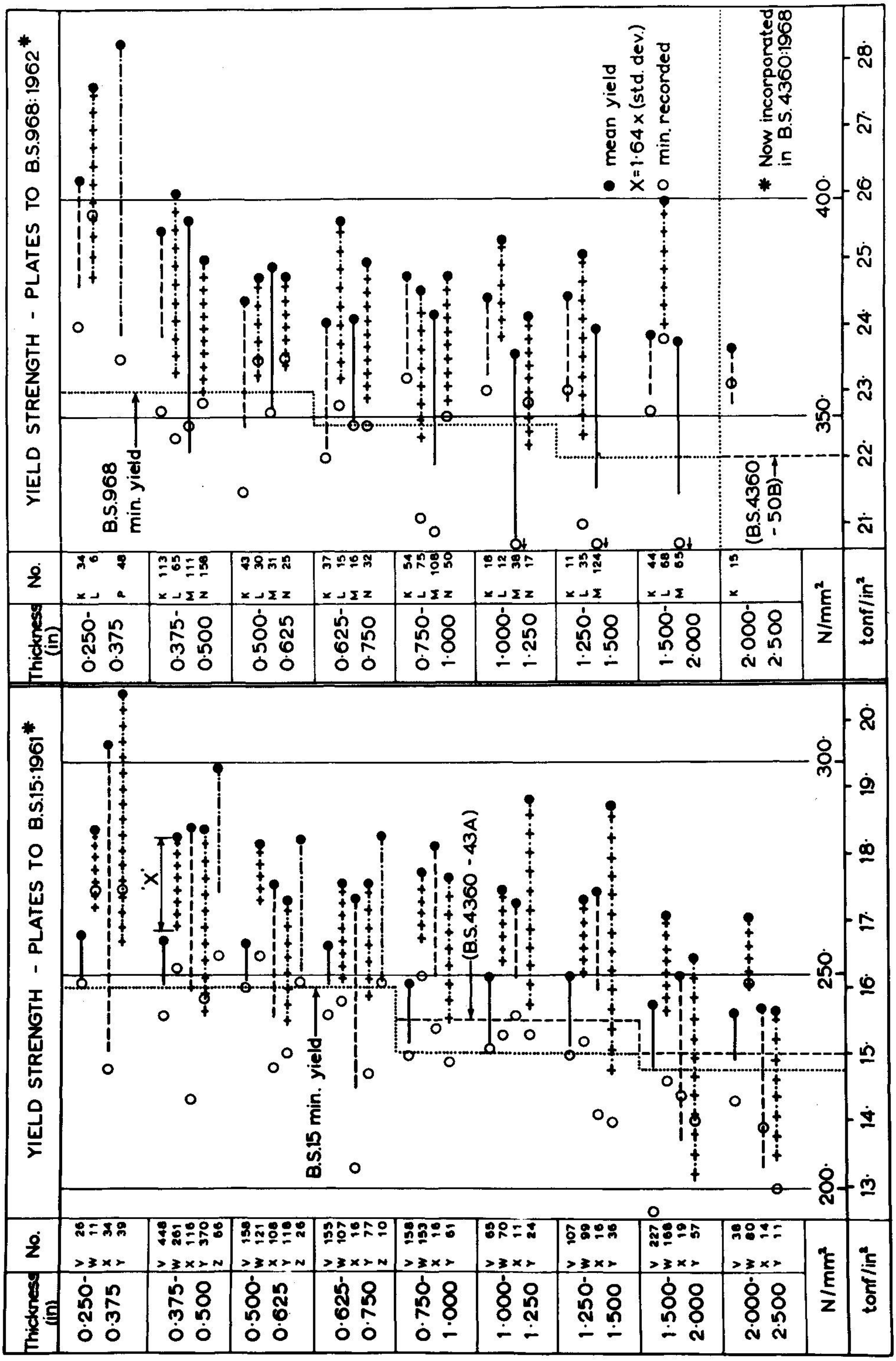
YIELD STRENGTH - PLATES TO B.S.15:1961*
YIELD STRENGTH - PLATES TO B.S.968:1962*
Thickness (in)
No.
B.S.15 min. yield
(B.S.4360 - 43A)
'X'
B.S.968 min. yield
(B.S.4360 - 50B)
● mean yield
X=1·64 x (std. dev.)
○ min. recorded
* Now incorporated in B.S.4360:1968
N/mm²
tonf/in²

FIGURE 1

2.2 High-Yield Steel Plates to British Standard 968:1962 The data have been supplied by the British Heavy Steel Association but unlike the B.S.15 steel relate to consecutive casts produced by each of the four mills over a short period of time and include all results. Summary statistics are given in Figure 1 and show that:

(i) as with mild steel plate, the variance is consistently lower for steel from certain mills than from others and this trend occurs for all thickness ranges

(ii) the 'characteristic' strength as defined above for steel from mill 'M' would be below the specified minimum value for all thickness ranges

As these samples relate to a relatively short period of time it is not possible to show whether the above trends are long term, but the fact that the same trends are shown by plates of all thicknesses, giving ten independent sets of samples, increases the confidence that can be placed in these results.

2.3 Universal Sections to B.S.15:1961 and B.S.968:1962 The source and method of sampling were the same as for high yield steel plates, and results with the same code letter refer to the same steel producing divisions of the British Steel Corporation (see Figure 2). Except for high yield sections with webs thicker than 1", the calculated characteristic strengths are in all cases greater than the specified minimum strengths. Comparison with Figure 1 shows that there is a much greater change in mean strength with thickness for sections than for plates and this is a direct result of the different rolling sequences required for the two products.

The statistics in Figure 2 relate to the yield strength of specimens cut from the webs of rolled sections, but as the overall strength of the section is more dependent on the mean yield strength of the flanges, the relationship between the mean web and flange strengths of different sections has been investigated. Figure 3 has been compiled from the results of commercial tests on about 200 separate sections, the specimens being cut from $\frac{1}{4}$ points in the webs and flanges. The results are classified according to the ratio of web/flange thickness, t/T.

The mean ratios of flange/web yield strength σ_{yf}/σ_{yw} increase with increasing t/T, but in addition σ_{yf}/σ_{yw} tends to unity as σ_{yw} decreases to the minimum allowable yield strength. Analysis of variance shows that this increase in σ_{yf}/σ_{yw} with decreasing σ_{yw} is statistically significant for both mild and high yield steel. At high values of σ_{yw}, σ_{yf}/σ_{yw} is significantly lower for some mills than

Mean Ratios of Flange/Web Yield Strength - B.S.15 Sections							
	Web Yield Strength - tonf/in^2						
t/T	15.25-16.25	17.25-18.25	18.25-19.25	19.25-20.25	20.25-21.25	21.25-22.25	22.25-23.25
0.55-0.60	-	1.05	0.93	0.91	0.89	0.87	0.87
0.60-0.65	0.99	1.00	0.97	0.93	0.92	0.90	0.87
0.65-0.70	-	-	-	-	0.93	0.91	0.90
0.70-0.75	-	-	1.01	0.93	0.98	0.96	0.93

Figure 3

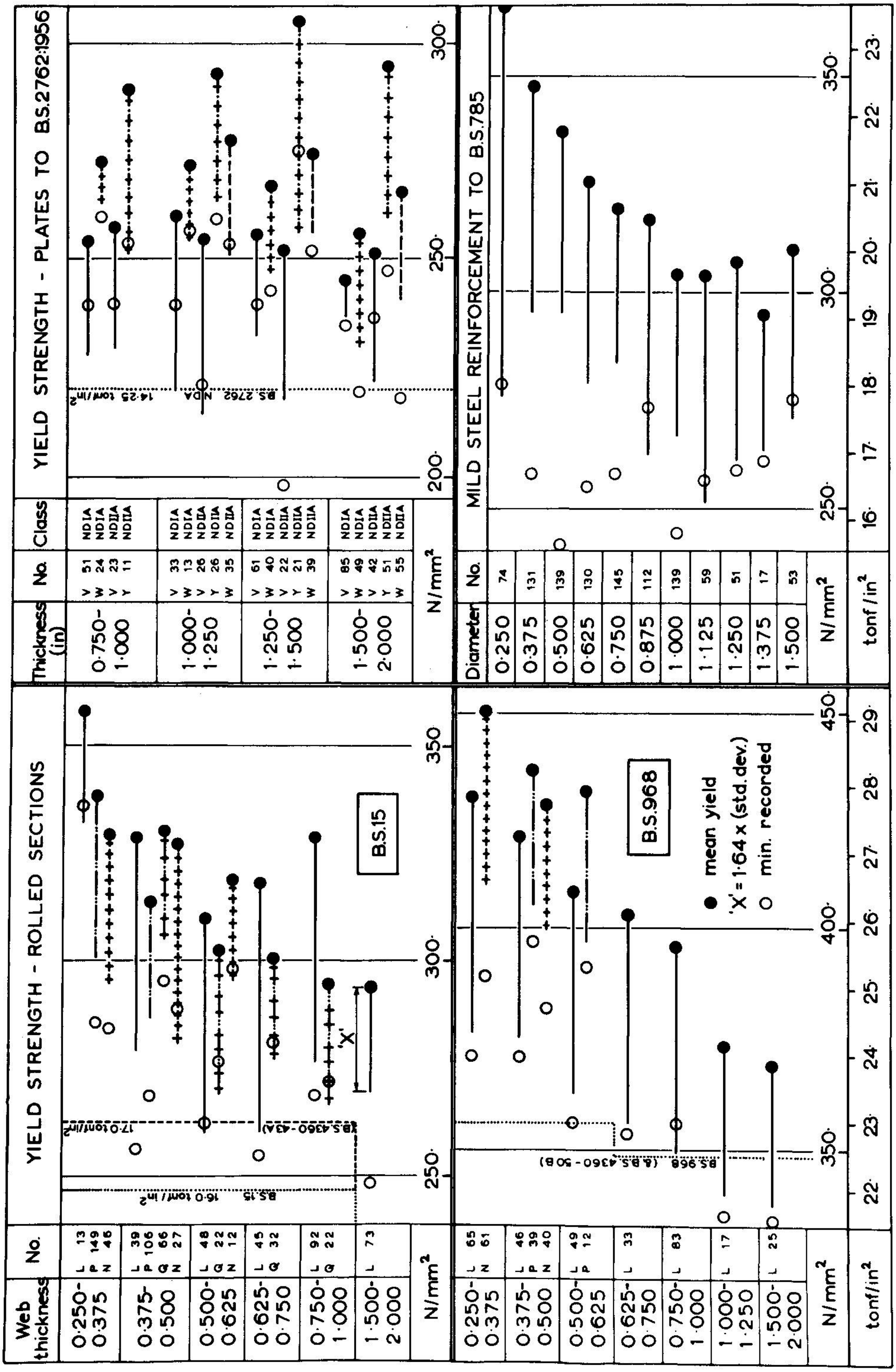
YIELD STRENGTH - ROLLED SECTIONS
YIELD STRENGTH - PLATES TO B.S.2762:1956
MILD STEEL REINFORCEMENT TO B.S.785
Web thickness
No.
Thickness (in)
Class
Diameter
B.S.15
B.S.968
mean yield
'X' = 1·64 x (std.dev.)
min. recorded
N/mm²
tonf/in²

FIGURE 2

for others (for all thicknesses), but at low values of σ_{yw} there are no significant differences and $\sigma_{yf}/\sigma_{yw} \doteq 1.0$ for all cases.

From the above, it follows that if any population of web yield strengths is normally distributed, then the distribution of the associated flange yield strengths will be negatively skew.

2.4 Notch Ductile Steel Plates to B.S.2762:1956 Figure 2 gives some statistics for plates having Charpy impact properties of 20 ft lb at 0°C and -15°C (corresponding to Classes NDIA and NDIIA respectively). As with the mild steel plates these data have been supplied by British Rail and results with the same mill code letter refer to the same steel mill.

2.5 Rectangular and Circular Hollow Sections to B.S.4:Pt. 2:1965 These data, comprising some 5300 test results on grade 16 and grade 23 steels, relate to the total production of a single manufacturer for the period 1964-1967 and are summarized in Figure 4.

2.6 Mild Steel Reinforcement to B.S.785 The results in Figure 2 for mild steel reinforcement relate to 1050 bars of different diameters supplied by an unknown number of mills and tested by an independent commercial test-house. Mean and characteristic yield strengths both show a marked decrease with increase in bar diameter, but as these results are based on samples which may originate from populations with different means and different standard deviations, the statistics are less meaningful than for other products studied.

3.0 STATISTICAL ANALYSIS

3.1 Normality The data discussed in Section 3 above have been checked for asymmetry and kurtosis to detect departures from the normal distribution. To eliminate errors due to the increase in mean strength with decrease in material thickness, only samples containing specimens of similar thickness have been selected from each size range for each material.

Results show that for plates, sections and hollow sections the distribution of yield strength tends to be positively skew for mild steel and negatively skew for high yield steel and that these tendencies are more pronounced for thin sections. Other experience has shown positive skewness for high yield steel in certain sections. It follows that characteristic strengths calculated on the assumption that the underlying distribution is normal will tend respectively to underestimate and overestimate the true values for these two types of materials.

In comparison with the above, it has been found that the distribution of ultimate tensile strength for the above samples is more normal than that of the yield strength, and that for all materials the distribution of ductility (as measured by the maximum elongation of tensile specimens) is always negatively skew.

Because of the large differences in the mean and variance of the yield strength of similar products rolled by different mills, data which have been selected at random from different mills to give a single sample will give poor estimates of the distribution of the combined populations from which they have been drawn. For the smaller thicknesses of mild steel plate it can be shown that the variance of the characteristic strength (based on a 95% probability of being exceeded and an assumed normal distribution) for different mills is smaller than the variance of mean strengths. It follows that the distribution of the combined populations for different mills will be positively skew, even if the distributions of the separate populations are statistically normal. This is a direct result of

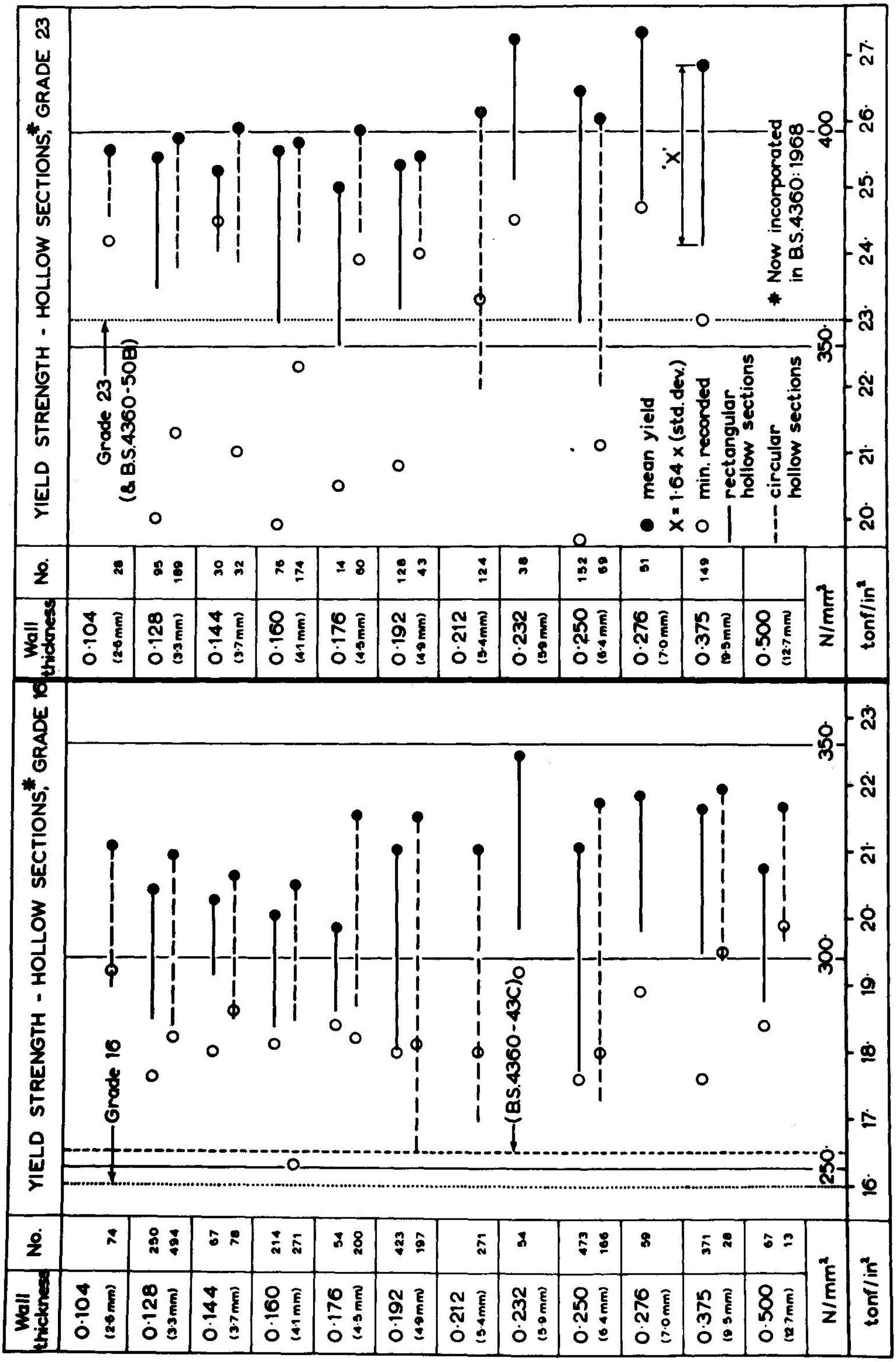
YIELD STRENGTH - HOLLOW SECTIONS,* GRADE 16
YIELD STRENGTH - HOLLOW SECTIONS,* GRADE 23
Wall thickness
No.
0·104 (2·6 mm)
0·128 (3·3 mm)
0·144 (3·7 mm)
0·160 (4·1 mm)
0·176 (4·5 mm)
0·192 (4·9 mm)
0·212 (5·4 mm)
0·232 (5·9 mm)
0·250 (6·4 mm)
0·276 (7·0 mm)
0·375 (9·5 mm)
0·500 (12·7 mm)
N/mm²
tonf/in²
Grade 16
(B.S.4360-43C)
Grade 23
(& B.S.4360-50B)
'X'
mean yield
X = 1·64 x (std. dev.)
min. recorded
rectangular hollow sections
circular hollow sections
* Now incorporated in B.S.4360: 1968

FIGURE 4

trying to achieve a yield strength in excess of a target minimum value (and corresponds to the zone between the ellipse and the straight line described by Leclerc[3]).

If, for example, two equal samples are selected at random from populations both having characteristic yield strengths of 16.0 tonf/in^2 but means of 17.0 tonf/in^2 and 19.0 tonf/in^2, the calculated characteristic strength of the combined sample will be 15.23 tonf/in^2, 0.77 tonf/in^2 below the true value.

The opposite is true, however, for the distribution of thickness of plates or sections rolled to a given nominal size, where both positive and negative tolerances are allowed. In this case, the combination of results from different sources tends to give a distribution which deviates less from normal than the underlying distributions.

3.2 Truncation There is a tendency for yield strength data based on mill test results to be subject to a certain degree of truncation for values less than the allowable minimum yield strength. With the existing system of batch testing steels (whereby only one test is carried out for every 'x' tons of steel rolled), it is unlikely that the material supplied by the steel mills is subject to the same degree of truncation, and a conservative design assumption is that the distribution of yield strength is not affected by the rejection of the material found to be below the minimum allowable value. However, with good quality control some manufacturers may be able to achieve a 100% cut-off.

Using a method suggested by Hald(4), the data discussed in Section 3 have been analysed assuming truncation at the minimum yield strength. For some samples with low characteristic strengths a significant degree of truncation has been detected and it follows that the true characteristic strengths in these cases are lower than the values calculated by simple analysis. This technique can be used only for samples which do not deviate from normal.

3.3 Variations in Mean Strength with Time In addition to random variations, the mean yield strengths of steel produced by a given mill may be found to change systemmatically with time, due to progressive changes in the manufacturing processes. Provided the data can be grouped according to time of manufacture, such changes can be detected by analysis of variance, and a reduction in the overall variance is then justifiable in calculating characteristic strengths.

4.0 THE EFFECTS OF VARIATIONS IN STRENGTH ON SAFETY

Johnson(5), Ferry-Borges(6) and others have shown how the probability of failure is highly sensitive to the assumed form of the distribution of variables. However, if it is assumed that the type of distribution of the yield strength of a certain grade of steel is the same for all manufacturers, a good estimate of the relative safety of the different products can be obtained by calculating the respective probabilities of failure P_f for a chosen distribution of loading. However, the results are still highly sensitive to errors in the estimated variance of the material properties.

Using the results obtained in Section 2 for different materials, and assuming a normal distribution of variables, the probability of failure of a simple flexural member (laterally restrained) subjected to a load with a coefficient of variation of 0.15 can be shown to vary as follows in Figure 5. For rolled sections, allowance has been made for the difference between web and flange strengths in calculating P_f and the safety factor γ , by using the mean correction factors given in Figures 4 and 5.

The comparisons are based on members designed with a safety factor of 1.7

Product	Thickness (in)	Mill	$\bar{\sigma}$	s	P_f	γ
B.S.15 Plates	0.375-0.5	Y	18.38	1.720	2.8×10^{-5}	1.65
	0.375-0.5	W	18.29	0.876	3.2×10^{-8}	1.79
	1.5 -2.0	Y	16.45	2.061	5.6×10^{-4}	1.54
	1.5 -2.0	W	17.08	0.924	3.2×10^{-8}	1.82
B.S.968 Plates	0.375-0.5	M	25.60	2.152	5.2×10^{-5}	1.63
	0.375-0.5	K	25.43	0.966	6.2×10^{-8}	1.77
	1.5 -2.0	M	23.76	1.415	3.4×10^{-6}	1.65
	1.5 -2.0	L	25.89	1.195	6.3×10^{-9}	1.84
B.S.15 Sections	0.375-0.50	Q	19.20	0.991	4.5×10^{-9}	1.93
	0.625-0.75	L	18.98	2.261	1.6×10^{-4}	1.79
B.S.968 Sections	0.25-0.375	N	27.42	1.585	1.5×10^{-9}	1.88
	1.5 - 2.0	L	23.85	1.257	3.4×10^{-6}	1.64
B.S.15 R.H.S.	0.144	-	20.29	1.085	1.3×10^{-10}	1.96
	0.250	-	21.07	1.700	8.0×10^{-8}	1.93
B.S.968 R.H.S.	0.232	-	27.26	1.306	5.4×10^{-9}	1.85
	0.250	-	26.46	2.116	5.4×10^{-6}	1.70

Figure 5

on the specified minimum yield strength of the material and have been selected to show some of the highest and lowest probabilities of failure for each type of steel, for mills with differing quality control. The comparisons do not include the effects of variables other than the basic yield strength of the material and the assumed distribution for loading, and in practice the probabilities will be higher due to other variables.

From the analysis of the variability of plate and rolled section thicknesses, it can be shown that the proportion of variance of the flexural strength of a simple steel member due to variations in cross-sectional dimensions is only about 1/10 of the proportion due to variations in yield strength. For such members, the variation in yield strength is thus the predominant factor governing safety (apart from load effects).

The safety factors γ shown in the last column of Figure 5 relate the load and the calculated values of characteristic strength, and are relative to a value of 1.7 for characteristic strengths equal to the minimum allowable yield strengths specified in existing British Standards. On this basis, the use of steel from different mills and the use of different types of section is equivalent to changing existing load factors of 1.7 to the values shown in the table (i.e. between 1.54 and 1.96).

ACKNOWLEDGEMENTS

This work has been carried out at Imperial College under the direction of Professor A.L.L. Baker and Dr. A.R. Flint and the author is extremely grateful for their advice and encouragement. The author is also grateful for the support of the Construction Industry Research and Information Association in financing this investigation and is indebted to the British Railways Board, the British Steel Corporation Heavy Steel Technical Committee and Stewarts and Lloyds Limited for making data available.

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SUMMARY

Variations in the strength of mild and high yield structural steels have been examined by analysing data from mill and other tests. Comparisons are made between similar steels produced by different mills and between similar steels of different rolled thickness. The results are interpreted in terms of probability of structural failure and recommandations are given for methods of evaluating characteristic strengths.

RESUME

Des variations de la résistance d'aciers de construction, doux et à haute limite élastique, ont été examinées en analysant les données d'essais de réception de laminoirs. Des comparaisons sont faites entre des aciers semblables produits par différents laminoirs et entre des aciers analogues de différentes épaisseurs de laminage. Les résultats sont interprétés en termes de probabilité de défaillance de la construction.

ZUSAMMENFASSUNG

Aufgrund der Auswertung von Walzwerk-Abnahmeversuchen wurden die Festigkeitseigenschaften verschiedener Fluss- sowie Baustähle mit hoher Streckgrenze untersucht. Aus verschiedenen Walzwerken stammende gleichartige Stähle sowie gleichartige Stähle verschiedener Walzstärke wurden jeweils miteinander verglichen. Der Verfasser erörtert die Ergebnisse im Hinblick auf einen etwaigen Bruch von Tragwerken.

IV

DISCUSSION LIBRE / FREIE DISKUSSION / FREE DISCUSSION

HERMANN BEER
Prof. Dr.
Graz

Mr. M.J. Baker mentioned in his contribution that, when considering the bending of I-sections, sufficient statistical data are not available at present to allow realistic calculations to be made on a truly probabilistic basis. This statement is valid to an even greater extent for all the other factors that exert an influence on the safety of structures.

Acting as an expert, I have had to deal, on many occasions, with failures and collapses of steel structures. The causes have always been: negligence on the part of the designer or constructor and more or less serious errors in the application of the principles of technology and of structural design and of the rules of calculation.

Summing up the causes of failures we can distinguish five main reasons for this occurrence:

1. Welding cracks due to inappropriate design and to the use of non-weldable steel and electrodes (brittle fracture), as well as incorrect welding processes.
2. Stability problems, with particular reference to lateral stability and restraint conditions.
3. Fatigue-cracks, which occur more especially in welded joints where there are unfavourable notch-conditions, under heavy traffic loads (i.e., gantry girders).
4. Errors in the assumptions made in regard to the structural system or in carrying out the static calculation and negligence during the making of the drawings.
5. Mistakes made during fabrication and assembling.

These errors and negligences cannot be avoided by statistical methods, but only by:

correct selection of the material
good design work
appropriate fabrication and assembling procedures.

Conscientious supervision during design and construction makes a very effective contribution to the safety problem. I agree that we should also put forward the probabilistic methods as a means of obtaining a more realistic image of the safety problem and of ensuring, finally, the consistent safety of different components and different types of structures, but we should not pretend that we are able, at present, to predict the probability of the collapse of buildings and bridges.

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Bemerkungen des Verfassers des Einführungsberichtes
Comments by the author of the introductory report
Remarques de l'auteur du rapport introductif

H. RÜSCH
Deutschland

Die Anwendung der in (1) zusammengefaßten Ergebnisse über die Streuung der Eigenschaften von Beton wirft zwei grundsätzliche Probleme der Sicherheitstheorie auf - nämlich die Frage nach dem Gültigkeitsbereich eines Sicherheitssystems und nach dem Mechanismus des Zusammenwirkens zwischen Sicherheitssystem und Qualitätskontrolle.

1. Übliche Sorgfalt und grobe Fahrlässigkeit

Es besteht kein Zweifel daß Fälle grob fahrlässigen Verhaltens, sei es im Entwurf oder bei der Bauausführung von keinem auf dem Prinzip der Wirtschaftlichkeit beruhenden Sicherheitssystem aufgefangen werden können. Voraussetzung jedes Sicherheitssystems ist, daß beim Entwurf und der Ausführung die "übliche" Sorgfalt angewendet wird. Fahrlässige Handlungen wie das Weglassen von Zement, die willkürliche Erhöhung des Wassergehaltes bei der Betonbereitung oder grobe Rechenfehler in der statistischen Berechnung können auch durch sehr große Sicherheitsfaktoren nicht aufgefangen werden.

Man muß darauf vertrauen, daß diese Fälle durch ausreichende berufliche Qualifikation und die übliche Sorgfalt des mit dem Entwurf und der Ausführung betrauten Personals verhindert werden. Werden aber diese Grundvoraussetzungen verletzt, so muß dies als Problem des Strafrechts und nicht der Sicherheitstheorie angesehen werden. Leider lehrt die Erfahrung, daß selbst Kontrollen grob fahrlässiges Verhaltens nicht völlig ausschalten.

Für eine auf wahrscheinlichkeitstheoretischen Grundlagen beruhende Sicherheitstheorie haben diese Festellungen wichtige Folgen. Im Falle der Betonfestigkeit betreffen sie im wesentlichen den bei der mathematischen Behandlung zu verwendenden Typ der Verteilungsfunktion. Es wäre aufgrund des Vorstehenden nämlich falsch, auch sehr kleinen Festigkeiten eine endliche Wahrscheinlichkeitsdichte zuzuweisen. Sie scheiden überdies schon deshalb aus dieser Betrachtung aus, weil solche Bauteile die Beanspruchung beim Entschalen bzw. Ausrüsten nicht überstehen. Wirklichkeitsnahe Verteilungsfunktionen für die Betonfestigkeit haben also bereits für Werte $x < \bar{x} - A$ und $x > \bar{x} + B$ die Wahrscheinlichkeitsdichte $\varphi(x) = 0$. Bei der Festlegung der unteren Grenze $\bar{x} - A$ ist zu beachten, daß sie sowohl durch ein Zusammentreffen ungünstiger Umstände bei der sonst mit üblicher Sorgfalt durchgeführten Betonbereitung entstehen können, als auch durch Fehler beim Einbringen des Betons. Es bietet sich an, im erstgenannten Fall die übliche Sorgfalt durch Vielfache der Standardabweichung, z. B. $\bar{x} - 3{,}5 \cdot \sigma$, gegen grobe Fahrlässigkeit abzugrenzen. Fehlstellen, z. B. Kiesnester, die bei üblicher Sorgfalt verbleiben, setzen die jeweils vorhandene Festigkeit auf einen Bruchteil herab. Hier erscheint es sinnvoll, grobe Fahrlässigkeit bei einem Wert $x < \bar{x}/3$ beginnen zu lassen.

Diese Einschränkungen machen eine mathematische Behandlung nur unwesentlich aufwendiger. Die Wahrscheinlichkeitstheorie liefert eine Reihe von geeigneten Funktionen, von denen allerdings jene den Vorzug verdienen, deren Entstehungsmodelle der Wirklichkeit am besten entsprechen. Ein Beispiel hierfür wird in (2) näher besprochen.

2. Sicherheit und Qualitätskontrolle

Die modernen Sicherheitssysteme deuten die charakteristische Festigkeit durchwegs statistisch als p %-Fraktile der Grundgesamtheit. Die mathematische Statistik lehrt uns, daß die bisher auf Baustellen üblichen Probenzahlen zum Teil nur sehr unsichere Aussagen über die vorliegende Grundgesamtheit zulassen. Andererseits ist vielfach nachgewiesen worden, daß die Versagenswahrscheinlichkeit außerordentlich empfindlich gegen Änderungen der Breite und der relativen Lage der Verteilung der Schnittgrößen und Querschnittsfestigkeiten ist. Wenn durch zweckmäßige Wahl der charakteristischen Werte für Festigkeiten und Lasten und der Sicherheitsfaktoren der Einfluß der unterschiedlichen Streuungen (Breite der Dichtefunktion) weitgehend ausgeschaltet werden kann, wie z. B. in (3) gezeigt wird, ergibt sich, daß Änderungen der Kenngrößen der Verteilung der Querschnittsfestigkeit durch Qualitätskontrolle beschränkt werden müssen. Grundsätzlich wird zu dieser Frage (4) Stellung genommen.

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IV

Remarques de l'auteur du rapport introductif
Bemerkungen des Verfassers des Einführungsberichtes
Comments by the author of the introductory report

J. LECLERC
France

Dans la préface du volume qui nous a été remis avant ce symposium, j'ai noté la phrase suivante :

"Le développement de la science et de la technique impose toujours davantage la recherche de principes généraux".

Je crois que les Sidérurgistes de tous les pays travaillent dans ce sens, dès lors qu'ils cherchent à mieux connaître les procédés d'élaboration et de transformation, pour permettre une meilleure utilisation de leurs produits.