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

---

*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).

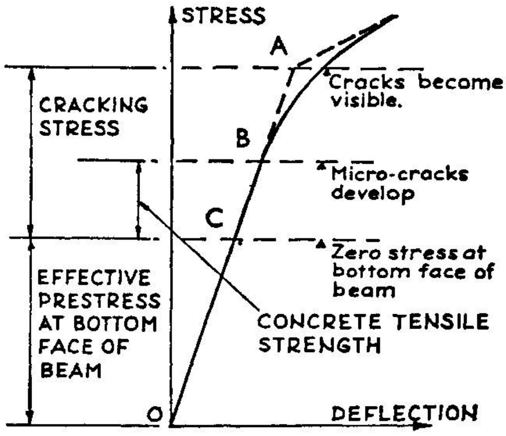


Fig. 1

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 (Eisenbeton-ausschuss) 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.

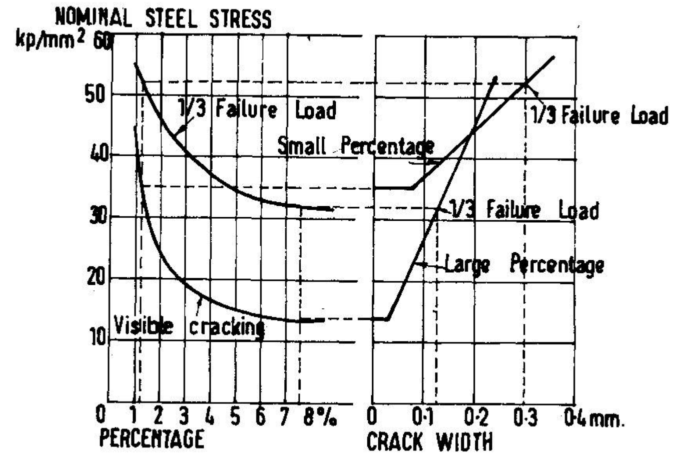


Fig. 2

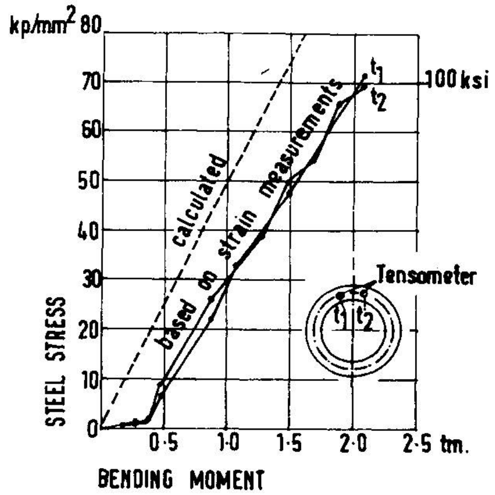


Fig. 3

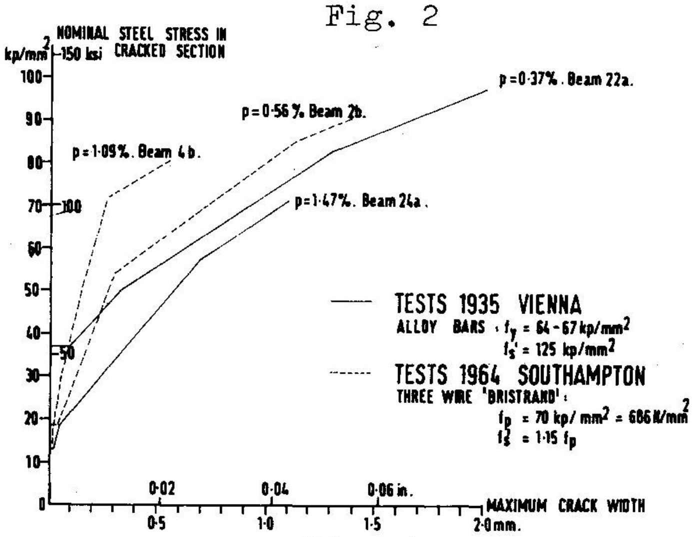


Fig. 4

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

BEAM NO.			1	2	3	4	5	6	7	8	9	10
SERIES A — Cover : 12·7 mm. CROSS SECTION· NOS 9–10, NOS 1–8	REINFORCEMENT	TYPE	Wire							Strand	Wire	
		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 — Cover : 25·4 mm. CROSS SECTION· NO. 10, NOS. 1–9		TYPE	Wire		Strand				Alloy Bars			Strand
		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

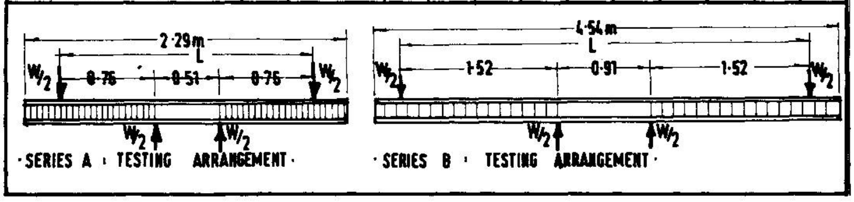


Fig. 5

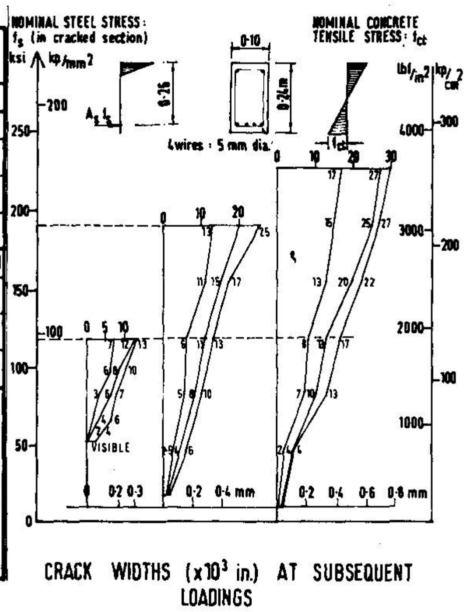


Fig. 6

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.

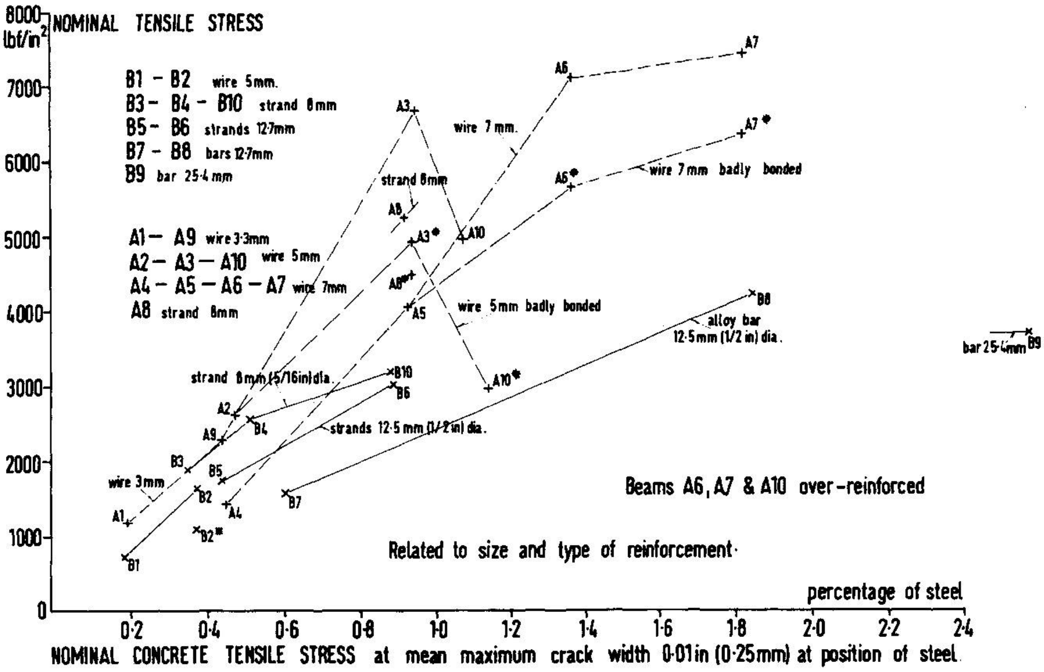


Fig. 7

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.

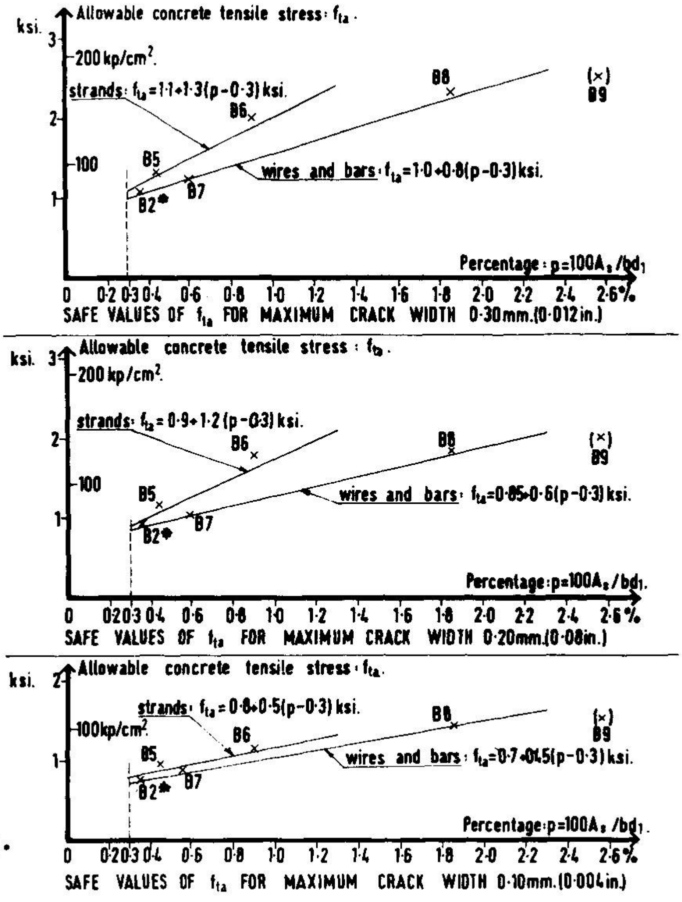


Fig. 8

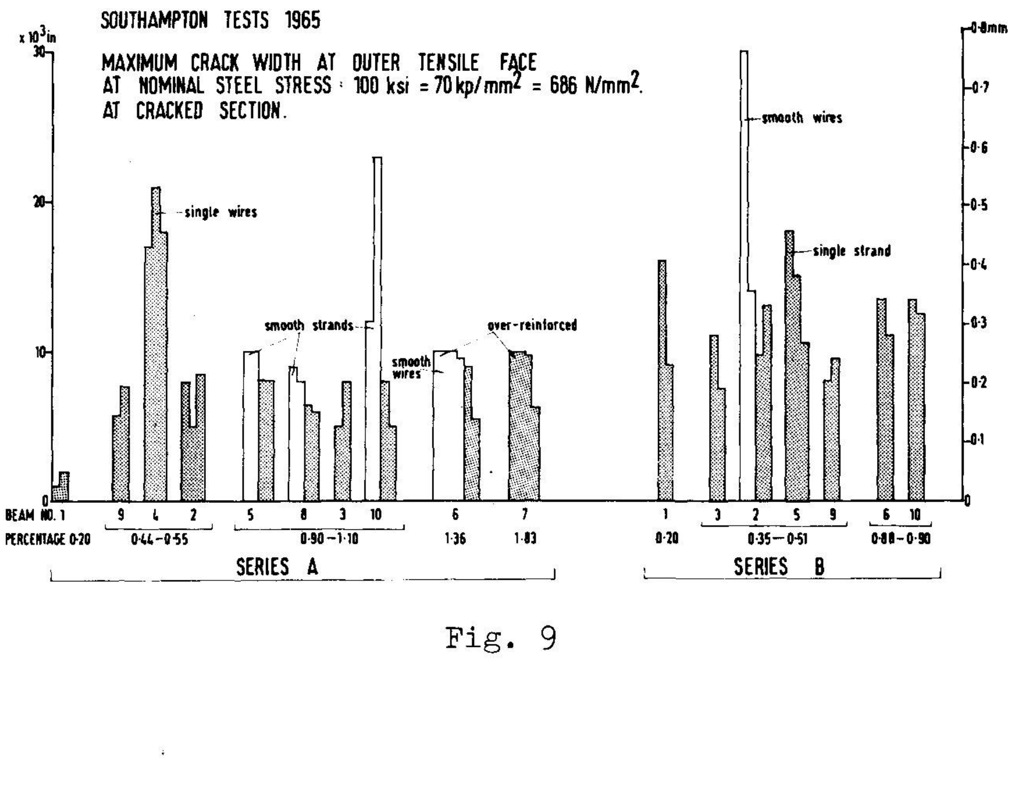


Fig. 9

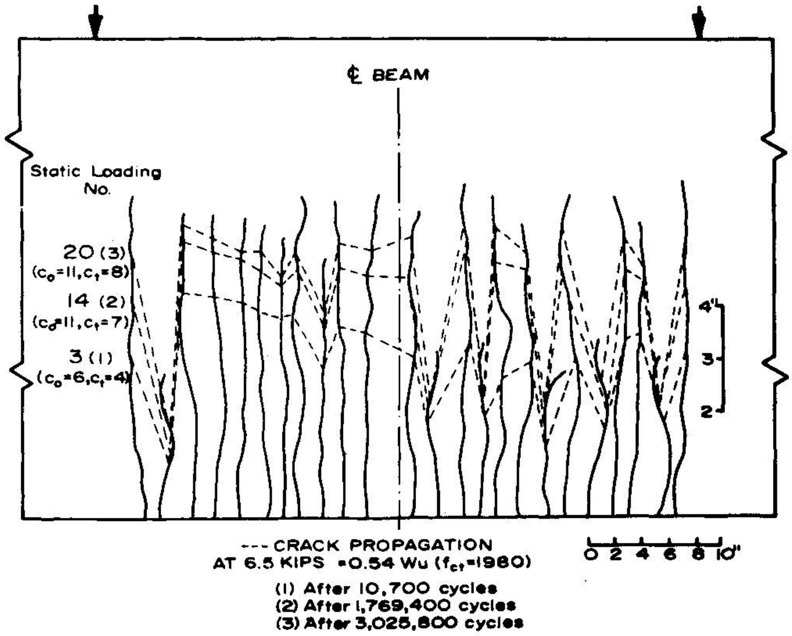
It is often propagated to measure all cracks, to determine the mean value of all widths and to compute the maximum crack width for each beam based on an assumed multiple of the standard deviation. In this case a very great number of fine cracks have to be measured which cannot be done with the same exactness 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, $\pm$ $2\frac{1}{2}$ %. 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.

BIBLIOGRAPHY

(1) R.H. Evans, "Extensibility and Modulus of Rupture of Concrete", The Struct. Eng., London 1946;

(2) P.W. Abeles & Chao-Hsiung Hu, "Flexural Microcracking in Unreinforced Concrete Beams", submitted to ACI;

(3) "Report on Prestr. Concr. Sleepers Tested as Simply Supported Beams", Concre. & Constr. Eng., 1947, Apr. & May;

(4) P.W. Abeles, "An Introduction to Prestressed Concrete", Vol. 1, London 1964;

(5) M. Roš, EMPA Report No. 162, 1950;

(6) P.W. Abeles, "The Practical Application of Partial Prestressing. Research on Cracking and Deflection under static, sustained and fatigue Loading" and P.W. Abeles & V.L. Gill, "The Behaviour of partially prestressed Beams, containing bonded, nontens. Strands & curved non-bonded Tendons", IABSE, 8th Congress, 1968;

(7) F.v. Emperger, "Die Rissfrage bei hohen Stahlspannungen und die zulässige Blosslegung des Stahles", Oesterr. Eisenbetonausschuss, No. 16, 1935;

(8) P. Abeles, "Ueber die Verwendung hochwertiger Baustoffe im Eisenbetonbau", Beton u. Eisen, 1935,Nos. 8 & 9;

(9) P. Abeles, "Die Rostgefahr von Eisenbetonkonstruktionen bei Rissbildung", Zement, 1937, Nos. 7, 8 & 9;

(10) P. Abeles, "Versuche mit Rechtecksbalken bewehrt mit besonders hochwertigem Stahl", Beton u. Eisen, 1937, Nos. 17, 18;

(11) P.W. Abeles & V.L. Gill, "High-strength Strand Reinforcement for Concrete", Concrete, London, April 1969;
(12) P.W. Abeles, Report on Tests, University of Southampton CE/8/66;
(13) P.W. Abeles, "Preliminary Investigations into Cracking of rect. prestr. Beams", CIRIA London, Techn. Note 7, 1969;
(14) P.W. Abeles, "Cracking and Bond Resistance in high-strength Reinforced Concrete Beams, illustrated by photo-elastic Coating", ACI Journal, Nov. 1966;
(15) P.W. Abeles, "An Introduction to Prestressed Concrete", Vol. 2, London 1966;
(16) P.W. Abeles, E.I. Brown & J.O. Woods, "Preliminary Report on static and sustained Loading Tests on Prestressed Concrete Beams", PCI Journal, August 1968;
(17) P.W. Abeles, E.I. Brown & J.W. Morrow, "Development and Distribution of Cracks in Rectangular Prestressed Beams during static and fatigue Loading", PCI Journal, October 1968;
(18) P.W. Abeles & F.W. Barton, "Fatigue Tests on Damaged Prestressed Concrete Beams" RILEM Symposium Mexico 1966;

(19) P.W. Abeles, E.I. Brown & J.M. Slepetz, "Fatigue Resistance of Partially Prestressed Concrete Beams to Large Range Loading", IABSE, 8th Congress, New York, 1968;
(20) P.W. Abeles, "The Design of Partially Prestressed Concrete Beams", ACI Journal, Oct. 1967;
(21) E.L. Kemp, F.S. Brezy & J.A. Unterspan: " Effect of Rust and Scale on Bond Characteristics of Deformed Bars", ACI Journal, Sept. 1968;
(22) E.J.M. Honigmann, "Witterungseinflüsse und ihre Raffung im Kurzversuch an Schleuderbetonmaststücken", Beton und Eisen, 1935 Nos. 19 & 20;
(23) P.W. Abeles, "Schleuderbetonmaste für Bahn-Fahrleitungen", Beton u. Eisen, 1935, No. 15;
(24) S. Soretz, "Korrosionsschutz im Stahlbeton und Spannbeton", Betonstahl in Entwicklung, No. 29, 1966.

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 Oberflächerbeschaffenheit 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.