Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 8 (1968)

Artikel: The practical application of partial prestressing. Research on cracking and deflection under static, sustained and fatigue loading

Autor: Abeles, Paul W. / Gill, Victor L.

DOI: https://doi.org/10.5169/seals-8839

The Practical Application of Partial Prestressing. Research on Cracking and Deflection under static, sustained and fatigue Loading

Application pratique de la précontrainte partielle. Etudes sur la fissuration et la déformation sous charges statiques continues et de fatigue

Die praktische Anwendung der teilweisen Vorspannung. Untersuchungen über Rißbildung unter statischer, bleibender und schwingender Last

PAUL W. ABELES
at present Visiting Professor in Civil Eng.
University Kentucky

including

The Behaviour of partially prestressed Beams, containing, bonded, non-tensioned Strands and curved, non-bonded Tendons

La tenue de poutres précontraintes partiellement, contenant torons adhésifs non-tendus et cambrés, et tendons non-adhésifs

Das Verhalten teilweise vorgespannter Balken mit schlaffen Litzen im guten Verbunde und nicht vermörtelten aufgebogenen Spanngliedern

PAUL W. ABELES
Research Fellow
University of Southampton

VICTOR L. GILL
Chief Civil Engineer
British Ropes Ltd.Doncaster

1. Introduction.

With the late General Secretary Dr P Lardy's assistance "Partial Prestressing" was included in the "Conclusions and Suggestions" of the Final Report (1),on the author's suggestion 20 years ago as acknowledged by the author in (2). Dr. Thürlimann (3) has summarized the advantages of partial prestressing. In Fig.1, the author's classification taken from (2) is shown, amplified by the FIP-CEB classification. For the latter a distinction between IIIA and IIIB is made. Class IIIA represents in the author's view, the most ideal solution with complete rigidity under "normal" service load, when fully prestressed, but exhibiting ductility when the limit state of service load is approached and exceeded, as was recently pointed out by the author in paper (4). Prof.Leonhardt now shares this view with regard to highway bridges (see pages 413-4 of (5)).

Prof.Thürlimann is not quite correct in stating on page 476 (3) that the author s suggestion of 1942 of prestressing the "total reinforcement" was applied in 1948. In fact, to his knowledge this system has never been used except for research. The system, introduced by the author at British Railways Eastern Region 1948-1962, relates to his proposal of 1940 of a "mixed reinforcement" comprising tensioned and non-tensioned prestressing steel. There is no need as stated on page 476 (3) that "The resulting tensile forces in the concrete have to be covered by an appropriate reinforcement".Tests on beams containing only tensioned steel have proved that visible cracking after the prestress has become ineffective solely depends on the concrete strength, the shape of the cross section, the bond efficiency and distribution of the steel (whether tensioned or not) . Prof.Thurlimann says on page 475(3) that with full prestressing according to equations (1) to (4) "the section exhibits a safety margin which is considerably above the specified one" when investigated at ultimate load. This applies only to the steel but not necessarily to the concrete compression zone. As ultimate load conditions are completely different from those in a homogenous section, even with a fully prestressed section the compressive zone may be too weak for ultimate load design although suitable for the conditions of a homogeneous section. For more particulars about "Partial Prestressing" see the Appendix of Vol.2. of the author's book [6].

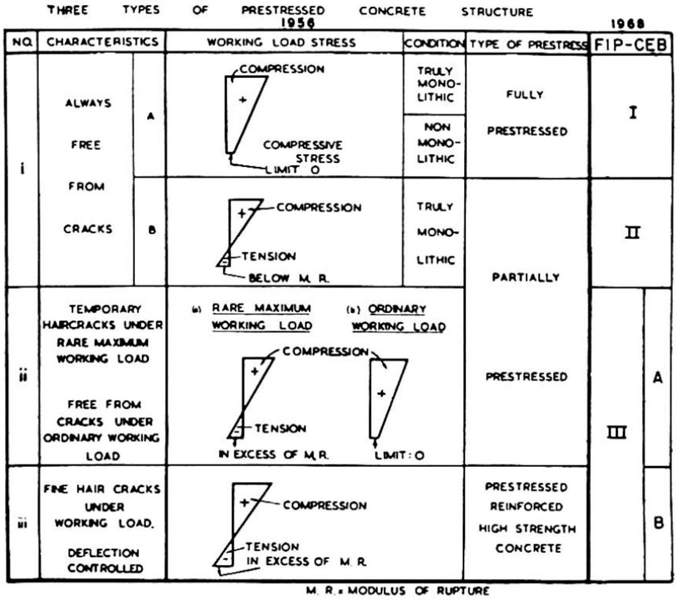


Fig.1.

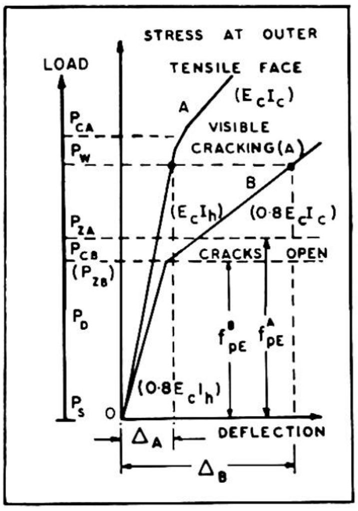


Fig.2.

2. Partially Prestressed Members Class II

With Class II, visible cracks will, in general, not occur, but microcracks develop. The author has reported at previous Congresses on the successful use of this type of partial prestressing by British Railways, Eastern Region, when tensile stresses of 650 to 750 psi (45 to 52.5 kp/cm^2) were permitted (i.e. 2/3 to 3/4 of the stress at which cracks become visible). The development of visible cracks has been avoided, as reported by the author[7], [8]. Strict supervision is necessary to avoid different behavior of structural members, as indicated in Fig.2. Beam "A" corresponds to the design, based on assumed maximum losses. If in a beam "B" the E-values of the concrete is only 80% of that in beam "A" and the effective prestressing force is less than assumed (either due to too low initial tensioning stress or to greater losses than assumed) the effective prestress f_{pE} is less. If shrinkage cracks have occurred before the prestressing force is applied, they may open when load P_{zB}, corresponding to the stress f_{pE} is exceeded. On the other hand, with beam "A" visible cracks will occur only when the flexural strength of the member f'_f is reached, which corresponds to the difference between the cracking load P_{cA} and the zero stress load P_{zA}.

The new trends of limit design load require probablistic considerations. With Class II, properties variations must be reduced to a minimum. This obviously requires strict supervision and preferably the use of random, non-destructive performance tests. More than 1500 such tests were carried out at British Railways Eastern Region between 1949 and 1962 as described in[6] page 550, and only a few rejections occurred at products from first jobs of prestressing works, when some mistakes in the application of the prestress had occurred or shrinkage cracks had developed before transfer of prestress. In one case, all 80 beams of a job were successfully tested.

3. Partially Prestressed Members Class III (IIIA and IIIB)

Class IIIA was, in principle, embodied already in the British Code of Practice CP 115 of 1951 where it is stated "Where the maximum working load to be considered is of temporary nature and is exceptionally high in comparison with the load normally carried, a higher calculated tensile stress is permissible, provided that under normal conditions the stress is compressive to ensure closure of any cracks which might have occurred". This allows a wide interpretation and "the temporary nature" may relate to the limit state of service load which may occur rarely but need not be instantaneous (e.g. snow load) or to abnormal instantaneous loads on bridges. The deflection at first loading and after repeated loading to the limit state, as well as due to creep, is illustrated in Fig 3. Small variations in production could be allowed for.

Prof. Thürlimann states under 5. "Method of analysis" that the "stress calculations can no longer be based on assumption of a homogeneous section." This is true, but there is no need for a stress calculation except for the fatigue range. It fully suffices to investigate (1) the limit state of collapse, thus ensuring a safety factor against failure and (2) to compute the required prestressing force. For Class IIIA no tensile stresses must occur at "normal" service load. For Class IIIB, the prestressing force P_E for a homogeneous section must be of such magnitude that at the limit state of service load neither too wide cracks occur nor the deflection becomes excessive, as will be discussed in the following.

Fig 4 illustrates these conditions. With IIIA the required minimum effective prestress f_{pE} in a homogeneous section must be equal and opposite to the tensile stress due to the normal service load, f_{sN}; there is no need to find out the stress conditions at abnormal load (limit state of service load). With IIIB the magnitude of the required minimum effective prestress f_{pE} can be obtained

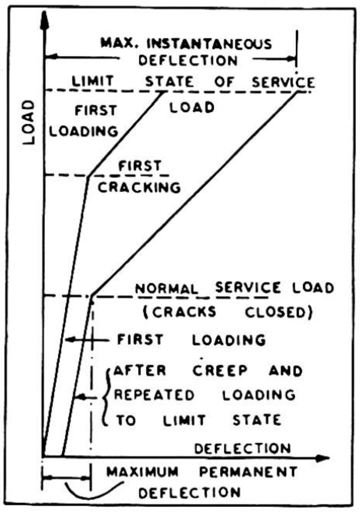


Fig.3.

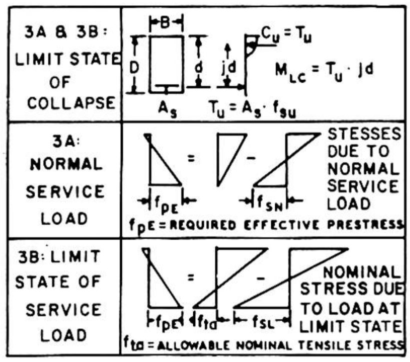


Fig.4.

as the difference between two fictitious stresses in a homogeneous section: f_{sL}, due to the entire service load, and f_{ta}, the nominal allowable tensile stress, which corresponds to the maximum permissible strain to limit the crack width. Members Class IIIA are also obtained if a skilful designer employs Prof. T.Y. Lin's method[9] of balancing bending moments.

While a structural member Class IIIA presents the most ideal solution, Class IIIB has also possibilities, but it is necessary in this case to study deflection and cracking at static, sustained and fatigue loading, the latter being mainly important for Class IIIA.

4. Cracking and Deflection at Static Loading

The maximum crack width for a definite strain depends on many parameters, varying with the shape of the cross section, the percentage, distribution and bond efficiency of the steel which latter again depends on its shape and surface conditions, and on the concrete strength. The author carried out tests on rectangular beams, containing non-tensioned prestressing steel, at the University of Southampton in 1965 to simulate the conditions after the prestressing force has become zero. Fig.5 shows deflection diagrams and crack widths at 3 loading cycles of a high strength concrete beam, containing 4 non-tensioned prestressing wires 0.2 in. dia.[10]. It is seen that the load deflection curves at the 2nd and 3rd cycles remained steep for a low load, corresponding to a homogeneous section. Simple and safe results for f_{ta} (the allowable nominal tensile stress in a homogeneous section) were obtained[11] for rectangular beams; e.g.

$f_{ta} = 800 + 1300\ (100p - 0.3)$ for round bars and $f_{ta} = 1000 + 2000\ (100p - 0.3)$

for strands, for maximum crack width 5×10^{-3} in. at the position of steel for concrete of a cube strength of approx.7000 psi ($420 kp/cm^2$), p being the percentage-ratio(s.Fig.6).

The choice of the suitable type of non-tensioned steel has also to be considered. Emperger suggested ordinary reinforcing steel as main reinforcement whereas the author pro--posed prestressing steel as non-tensioned steel and never used more than half the entire steel as non-tensioned reinforcement.

With high strength steel, the deflections become relatively large though the cracks remain narrow, whereas with mild or medium strength steel the deflection is greatly reduced, but more space is required for placing the steel, and the construction becomes less economical, as pointed out by Prof.Zerna (12). Prof.Kani (13) has referred to the shortening due to shrinkage and creep which produces a compressive force in the steel; he states that an opposite equal tensile force must occur in the concrete, thus reducing the prestressing force similar to the behavior in columns. This would mean that the greater the cross sectional area of the non-tensioned steel, the greater becomes the compressive force in the non-tensioned steel, though the amount of creep and shrinkage would be reduced by the increase of the steel section. Comparative tests, however, have indicated that there is very little difference, if any, between the loads at which cracks become visible, though microcracks seem to occur earlier with the larger reinforcement of mild steel. A recent study by Prof Shaikh (14) and Branson indicates that there is no difference in visible cracking.

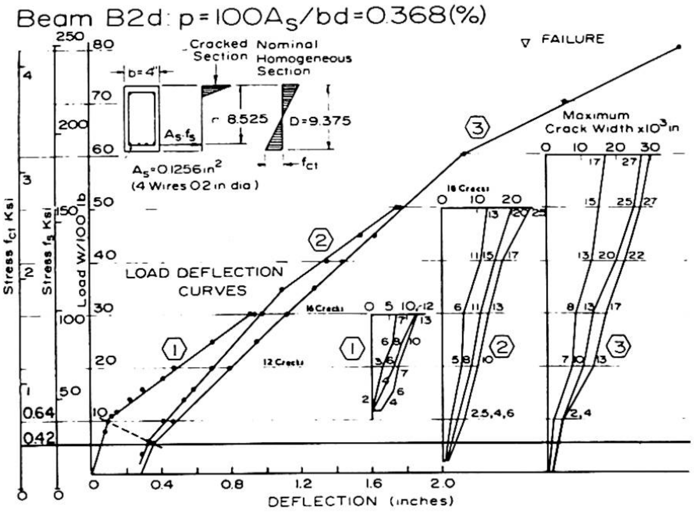
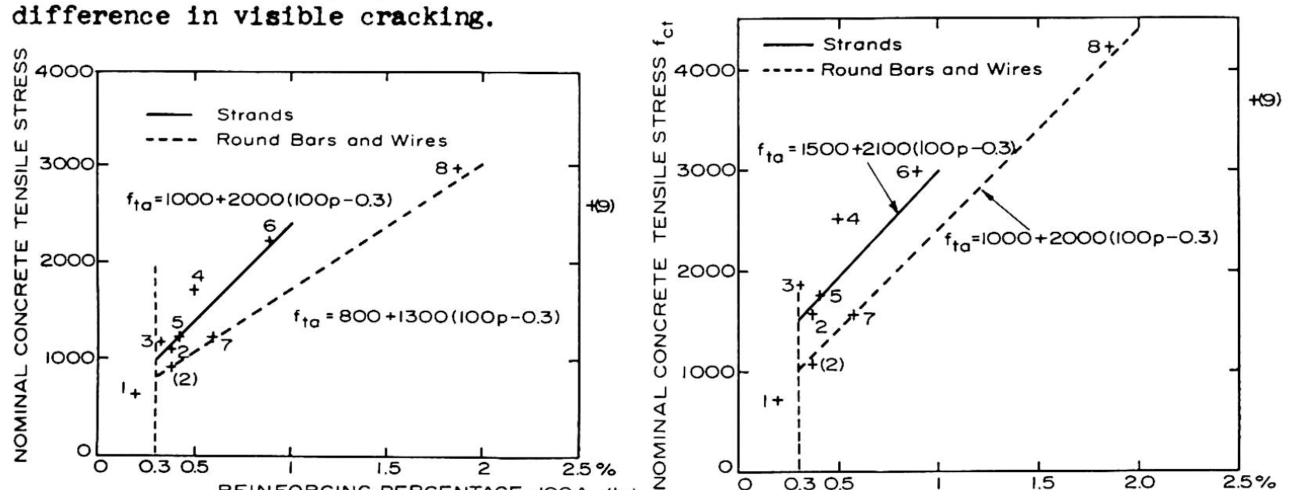


Fig.5.



(a) Maximum Crack Width 0.005 in.

Fig.6.

(b) Maximum Crack Width 0.01 in.

Fig.7 shows results of comparative tests carried out at the University of Kentucky 1967 (15). In this case, in addition to two alloy bar tendons, 3 types of non-tensioned steel were used; prestressing strands (AS), high strength steel(AH) and mild steel bars(AM) of equal nominal yield force. With all 3 beams failure occurred at almost the same load and agreed well with the predicted ultimate load, based on the actual position of steel. There were only slight differences in cracking load. In the beams Fig 7 the steel was displaced and thus in the uncracked section the beam AS, containing strands with the smallest percentage, was the stiffest, although AM ought to have been the stiffest. The Table I shows number and width of cracks.

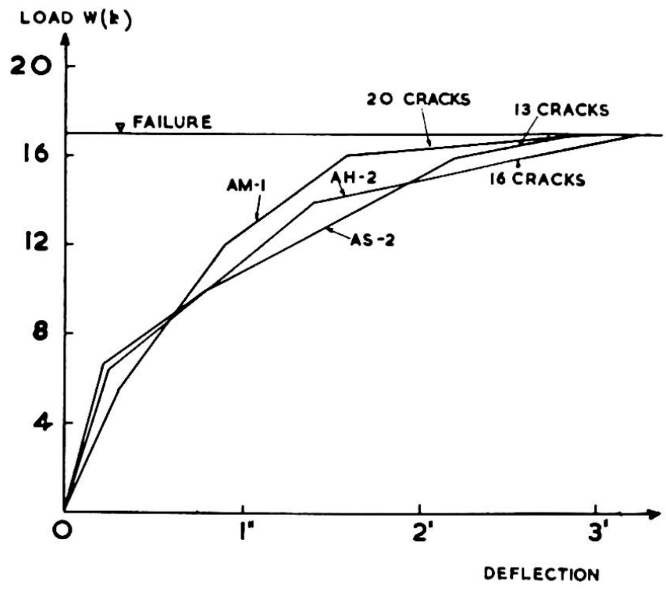


Fig.7.

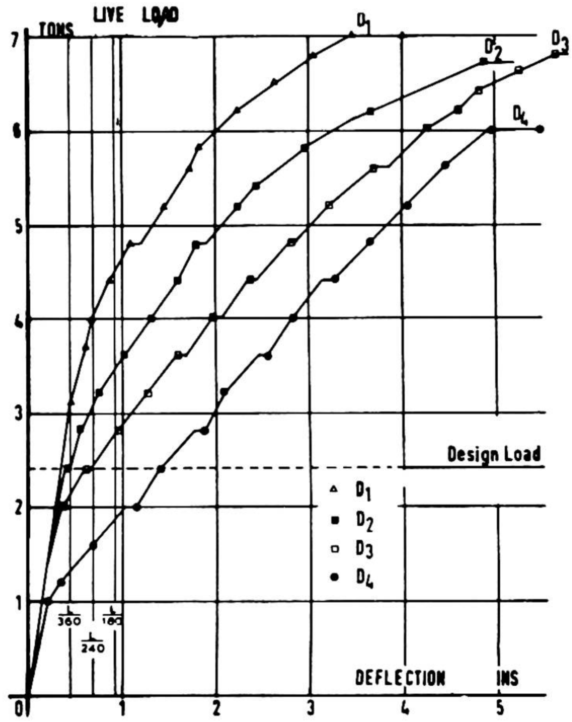


Fig.9.

CRACKS AT LOAD W=8K				
BEAM TYPE	CRACK NUMBER	MAXIMUM CRACK WIDTH x 10^{-3} IN AT OUTER TENSILE FACE	AT LEVEL OF REINFORCEMENT	AT LEVEL OF TENDON
AM	15	4	3	3
AH	10	6	3	2
AS	8	16	4	3

Table I.

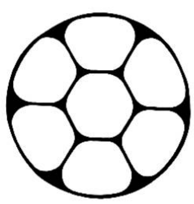


Fig.8.

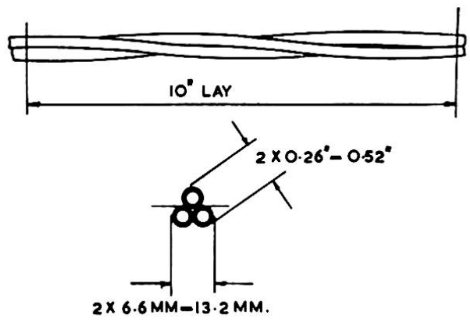


Fig.10.

In addition to the EI values, also cracking affects deflection. Reference is made to Fig P.11 in (6)(p 630), comparing the deflection of two rectangular beams, one containing a single deformed bar and the other two non-tensioned prestressing strands of slightly smaller area, both placed at the same depth. In spite of the slightly larger steel area the former beam had a greater deflection, in consequence of fewer and wider cracks.

In the following, an example is shown of the use as non-tensioned steel of the "Dyform" prestressing strand of especially high strength (Fig. 8). This strand is drawn through a die, which produces a relatively smooth cylindrical surface, but it showed very satisfactory bond characteristics. The following data have been taken from a thesis by Dr Dave and have been kindly supplied by Dr Bennett, of Leeds University(16). Rectangular beams 5 in. wide and 8 in. deep containing 3 "Dyform" strands 5/16in. dia. $1\frac{1}{4}$ in. above the tensile face were tested for a span of 14 ft. with two point loads at the quarter points. Fig. 9 illustrates the load deflection diagrams of 4 beams: D1 in which all 3 strands were tensioned, D2 with two tensioned and one non-tensioned strand, whereas in D3 one was tensioned and two remained non-tensioned, D4 relating to a beam in which all 3 strands remained non-tensioned. This figure has been included, because it clearly shows the advantages and limitation of partial prestressing using super-high strength non-tensioned steel

In view of the excellent bond and crack performance of beams containing non-tensioned prestressing strands a new three wire strand called Bristrand 100 was introduced by British Ropes Ltd. (see Fig.10). This steel has a proof stress of 100 ksi (70kp/mm^2) and a minimum strength of 120 ksi (84 kp/mm^2), the diameter of the individual wires being 0 263 in (66mm). It was assumed that this type would be suitable both as high strength reinforcement for ordinary reinforced concrete and as non-tensioned bonded steel in partially prestressed concrete, representing a medium strength between ordinary high strength steel and prestressing strands. Tests on beams containing such Bristrands are described in the following section. The actual strength of the steel was 139 ksi(103 kp/mm^2) and the Modulus of Elasticity 27 x 10^6 kp/cm^2).

6. The Use of non bonded Tendons.

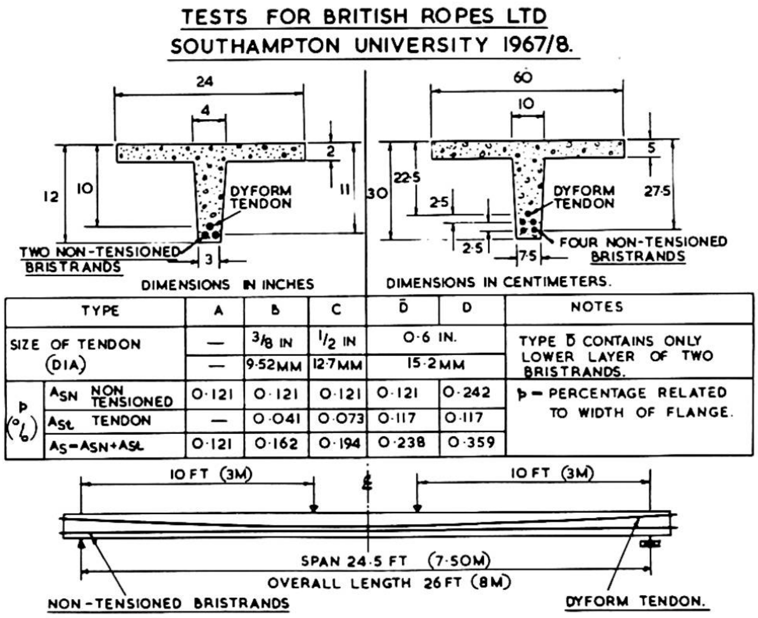
Non-bonded or badly bonded, post-tensioned tendons behave less satisfactorily than well bonded tendons. Only a few wide cracks occur and the flexural resistance is reduced. However, there are advantages, as the pressure grouting of the tendons can be dispensed with and thus a situation is avoided at which efficient grouting under adverse conditions may become rather difficult and unreliable. Moreover it is possible to readjust the prestressing force at a later date to offset the losses due to creep and shrinkage. Obviously a special corrosion protection is essential.

By the use of partial prestressing the great advantage of good bond can be kept and the disadvantages, mentioned above, can be avoided by the use of well bonded non tensioned steel, placed closely to the tensile face. This was shown by Prof Burns(17) who had carried out tests to prove that continuous beams, containing non-bonded tendons in conjunction with bonded medium strength steel, behave much better both in bending and in shear than expected by the Codes. The author has proved that the expected ultimate resistance for bonded steel was obtained by the tests Fig.6 described before This figure was previously used only to show by comparison the effect of different types of non-tensioned steel. It was not pointed out that in this case the tendons were non-bonded, because the crack distribution was very satisfactory and the ultimate load reached corresponded very well with the computed values based on the yield points and proof stress of the non-tensioned steel and the proof stress of the alloy bars.

In the following, tests are briefly described, carried out by the authors at the University of Southampton between Oct.1967 and Jan.1968, in which the newly introduced Bristrand 100 (Fig.10) was used as bonded,non-tensioned reinforcement. Fig.11 illustrates the test programme and Fig.12 is a photograph of a beam under test. Two beams of type A were not prestressed. Four beams each of types B,C, & D were originally planned, two grouted and two non-grouted. Since there was hardly any noticeable difference between these two types with beams B and C, one beam of the original type D was differently reinforced as type $\bar{D}$. The stress strain diagrams of the various sizes of Dyform Strands are seen in Fig.13 which also contains the size relating to the tests Bennett-Dave (Fig.9).

Fig.14 shows the deflection diagrams for the applied loads of all beams all of which were under-reinforced. There was great uniformity in the maximum load and deformation, except for one beam D, illustrated by a dotted line. With all other beams failure did not occur, but the tests had to be terminated in consequence of too great deflections. The recovery on removing the load was between 80 and 90%, although in some cases horizontal cracks developed during removal of the load in consequence of the very large change in strain.

The beam with the dotted deflection diagram failed by crushing of the concrete at a bending moment of approximately 75% of the maximum of the other two beams D and it was noticed that there was a pocket in the flange with poor compaction. This is a good example of the importance of strict supervision. The 14 beams described were cast in a civil engineering workshop using ready mixed concrete. The two authors supervised either jointly or separately, casting, prestressing and pressure grouting. Unfortunately, when the two beams D13 and D14 were cast, the supervising engineer had to leave shortly before completion of the casting. Thus beam D13 (D) was well cast but the flange of beam D14 evidently did



TYPE		A	B	C	D̄	D	NOTES
SIZE OF TENDON (DIA)		—	3/8 IN	1/2 IN	0·6 IN.		TYPE D̄ CONTAINS ONLY LOWER LAYER OF TWO BRISTRANDS.
		—	9·52MM	12·7MM	15·2MM		
p (%)	A_{SN} NON TENSIONED	0·121	0·121	0·121	0·121	0·242	p – PERCENTAGE RELATED TO WIDTH OF FLANGE.
	A_{SL} TENDON	—	0·041	0·073	0·117	0·117	
	$A_S=A_{SN}+A_{SL}$	0·121	0·162	0·194	0·238	0·359	

Fig.11.

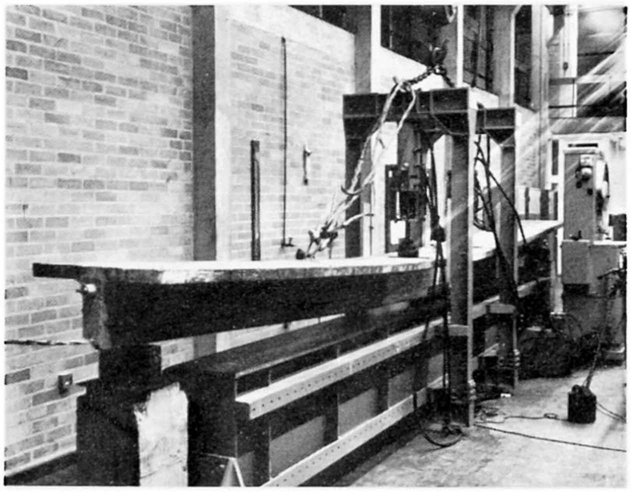


Fig.12.

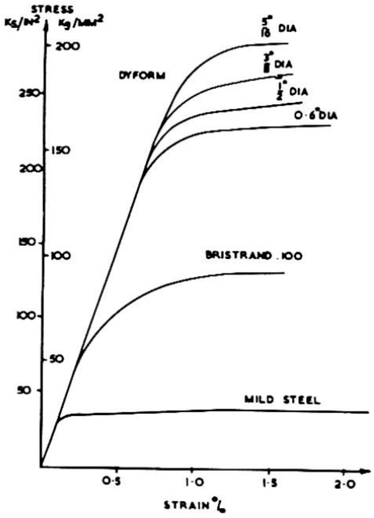


Fig.13.

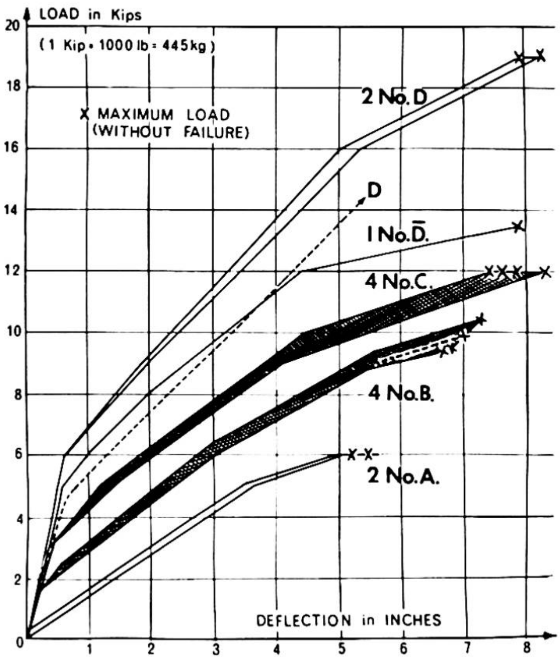


Fig.14.

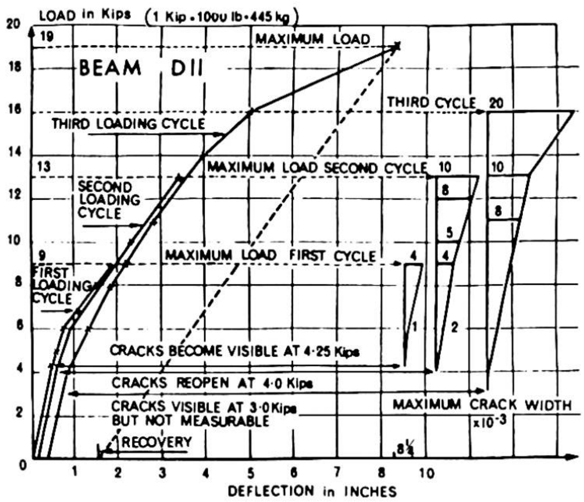


Fig.15.

not have the concrete properly vibrated and this poor concrete was the cause of the early collapse.

More details and particulars about electric resistance strain gauge readings and analysis of the test results will be published in a paper by the two authors later. An example is shown in Fig.15 of the deflection diagrams at 3 load cycles of beam D11 containing a non-bonded tendon. Also the maximum crack widths are given. The maximum loads generally agreed with the values obtained for the guaranteed minimum strength of the Bristrand 100 and the proof stress of the Dyform tendons which were positioned further away from the tensile face.

6. The Effect of Sustained Loading.

This is important with members Class IIIB. Fig.16 shows the result of a test carried out by British Railways (Research Dept. Derby and Chief Civil Engineer Eastern Region) on a rectangular beam 8"x 12" (20x30cm) containing two layers of wires 0.2 in. (5mm) dia. of a span of 13 ft. 6 in. (4m) with 2 central loads 3 ft. 6 in. (1.06m) apart. The prestress at transfer was 2700 psi (189 kp/cm^2) at the outer tensile face The loading carried out at Derby Station commenced only after approx. 2½ years, when the initial camber had more than doubled. The beam was subjected to a load of 50% of the static failure load (S.F.L.) of a companion beam, when microcracks occurred, the nominal tensile stress being 880 psi (62kp/cm^2). The load was sustained for almost 3 years and then increased to 80% of the S.F.L., when 5x10^{-3}in. (0.125mm) wide cracks developed which increased to 3 times the size during more than one year, when the test terminated; but the deflection had increased only by 50% during this time. More particulars are seen in Fig. P.14 of [6], which does not, however, show completion of the test. The beam with permanently open cracks of 15x10^{-3}in. (0.48mm) was exposed to the highly polluted surroundings of Derby Station, but the corrosion of the well bonded wires was relatively small.

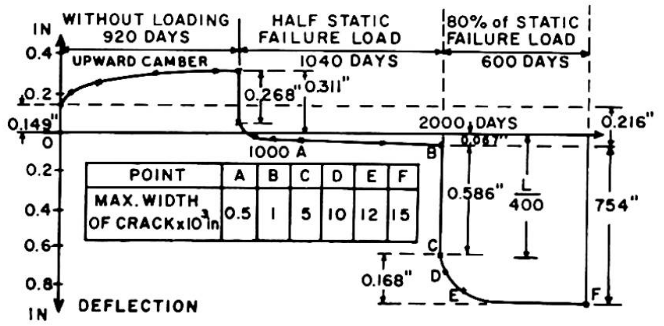


Fig.16.

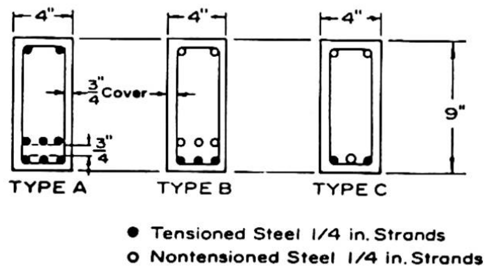


Fig.17.

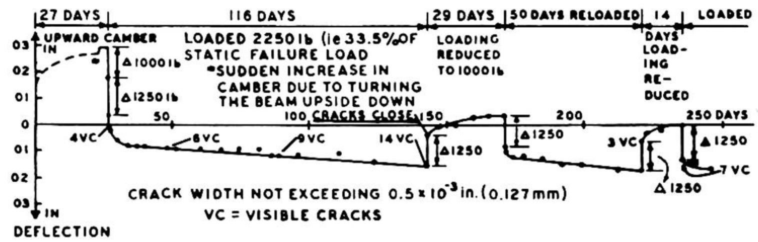


Fig.18.

Prof. Brown of DUKE University arranged together with the author comparative static and sustained loading tests to be reported in paper [18]. Fig. 17 shows cross sections of the beams types A, B and C and Fig.18 illustrates the results of sustained loading and unloading on beam CL1, the letter L indicating semi-light weight material. Span and loading points were as with the beam Fig.16, but the load was applied from below. The load applied at an early age was of such magnitude that microcracks developed. For more particulars see paper (18).

7. Cracks and Deflection after Fatigue.

The author reported in Cambridge[19] on tests carried out at Liège in 1951 that 3 million load cycles within definite stress ranges, during which cracks opened and closed, did not affect the failure load in a subsequent static test. Full serviceability and change in slope of deflection due to fatigue after cracking was discussed in Lisbon [20]. The crack width may become almost 3 times as large as the initial value at approaching fatigue failure, as shown in P.9[6]. Prof Ekberg and Assoc.[21] have shown that the stress range within the Goodman-diagram governs fatigue failure. However, also the width, extent and distribution of the cracks greatly influence fatigue behavior. Members with well bonded and distributed steel, having the same percentage of steel and being subjected to the same stress range in the steel, will have a longer fatigue life than members in which the steel is concentrated and/or badly bonded. It has been found that occasional overloadings are of no influence on the fatigue resistance under working load. Dr. Brown of DUKE University arranged fatigue tests on a new MTS machine, which allowed to study this and other problems. First experiments were carried out in 1966 on two beams (22) which had already failed in a static loading test by yielding(18), but were only slightly damaged by spalling off some edges in the compression zone. Almost complete recovery took place on immediate removal of the load. Fig.19 depicts the results on beam BL1 (see also Fig. 17). The static loading had been carried out in 3 cycles (curves 1,2 & 3); W_o is the load at which the effective precompression at the tensile face became zero. The applied static failure live load of 10.5 k. agreed quite well with the calculated value

Cycling loads over 10 different ranges were applied (I-X), with 6 intermediate static load deflections (A-F). With I, the range was 14% and with II-VII it was 19% of S.F.L., gradually extending to an upper limit of 76% with approx. 80,000 cycles at each range The following ranges VIII and IX with 20,000 and 29,000 cycles respectively extended over a 25% larger range, the displacements increasing greatly as indicated in the figure. The upper limit was increased at IX and further at X to 85% and 90% respectively of the S.F.L. Fatigue failure occurred due to fracture of one wire of one strand after 143 cycles of X between 52 and 90% of the S.F.L. when the beam collapsed, the entire fatigue test comprising 605,000 cycles. For further particulars see paper (22).

Based on further fatigue tests a joint paper (23) was presented in which it was reconfirmed that occasional overloadings (e.g. 20,000 cycles) did not affect the fatigue resistance (2 million cycles) within a lesser stress range, although in the latter the cracks opened and closed a million times. Note that a weekly abnormal loading would only amount to 5200 cycles in 100 years.

In the following, only Fig.20 is shown from this paper to illustrate the test on beam AL2 which was loaded until fatigue failure occurred. The beam was first subjected to 105,000 cycles between 14 and 36% S.F.L. until previous microcracks just became visible. Afterwards, 307,700 load cycles were applied between 30 and 70% of the S.F.L. of a companion beam. Failure occurred by fracture of 7 wires of the central lower strand and 2 wires of one of the outer lower

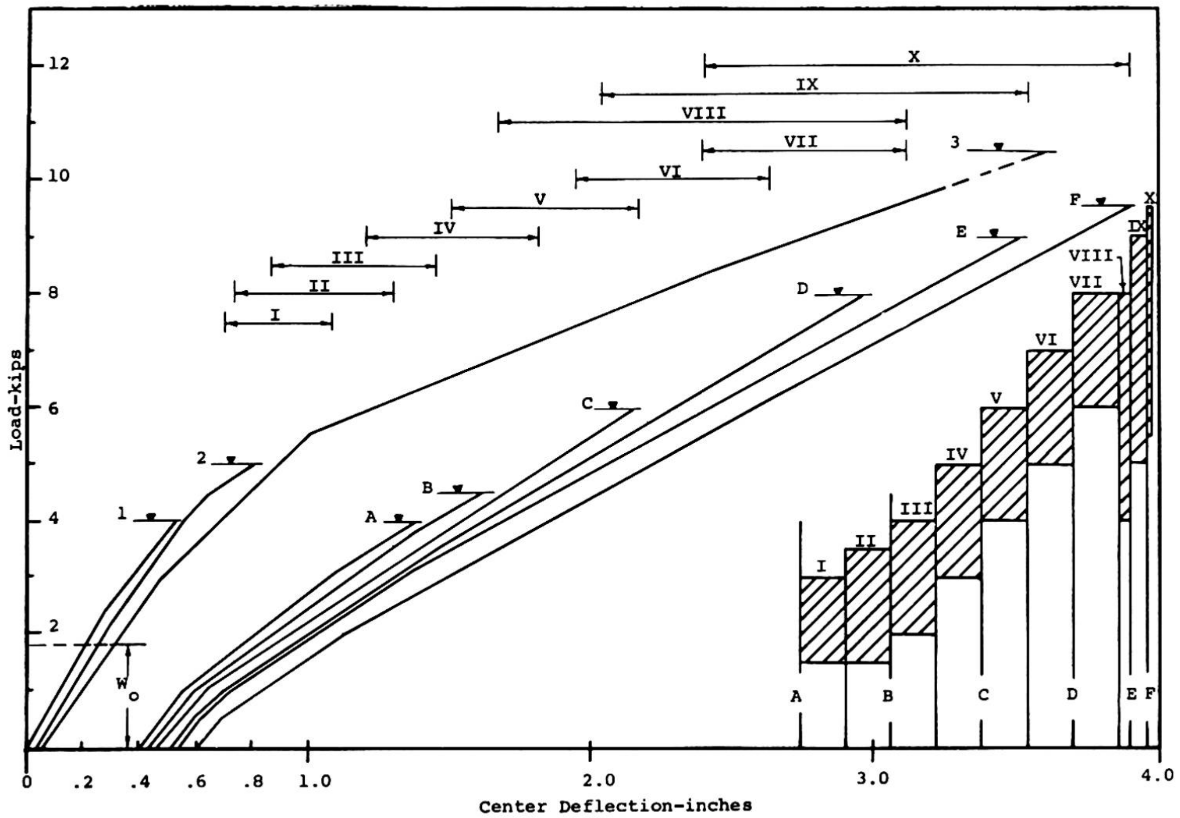


Fig.19.

strands. Fig.20 also shows the effect of fatigue on deflection and cracking; diagram 3 refers to a loading, before the large range fatigue loading commenced, No. 4 to a loading after 5,500 cycles, No.10 to a loading after further 222,200 cycles, while No.11 relates to a static loading after fatigue failure. This very important test result has indicated the importance of studying the fatigue resistance of prestressed concrete over large ranges to obtain L-N curves (i.e. load versus the number of cycles), the lower limit of 30% relating to dead load. In subsequent tests at DUKE University the strains were measured by electrical resistance strain gauges. Much research has still to be done, as is pointed out in the joint paper [24] and it is hoped that it will be possible to continue the further tests required to clarify these problems.

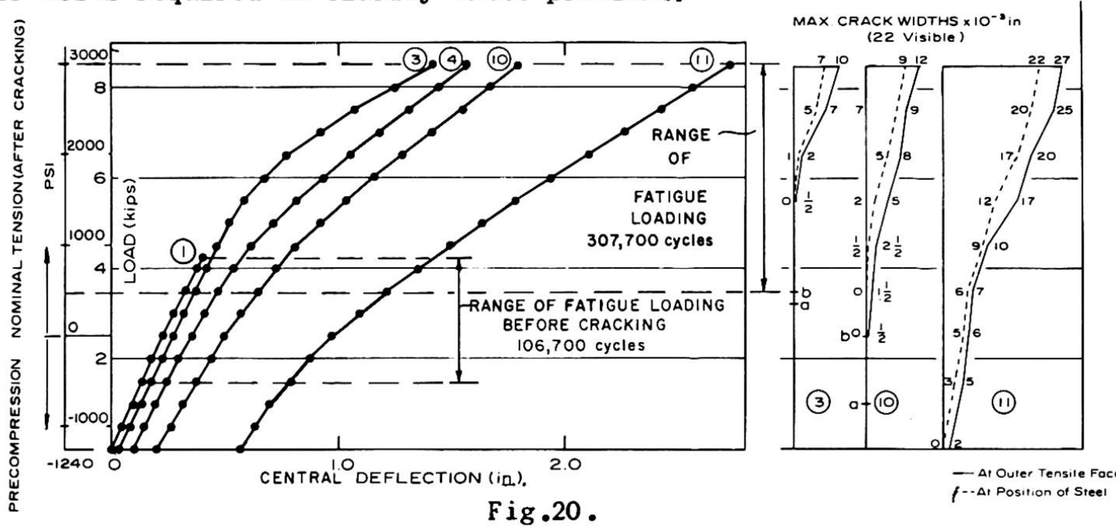


Fig.20.

Reference may be made to the importance of obtaining satisfactory Goodman-diagrams for steel, based on S-N curves. Papers by Warner & Hulsbos [25], Tide & Van Horn [26], Hilms & Ekberg [27] may be mentioned. As soon as reliable S-N curves for prestressing strands and wires are available, (which have to follow the law of probability and are to be based on safe values), it should be possible to obtain rather safe L-N curves for prestressed concrete beams of substantial depth. In this case faults in workmanship cannot greatly affect dimensions and position of steel, whereas with very small members great scatter and thus substantial variations in test results may occur, as experienced by Prof.Venuti [28]. Dr.Dave-Dr.Bennett found a relatively great fatigue resistance by single tests on various types of their differently prestressed members, as will be found in [16] when it appears. They have also established a design method for determining the steel stresses in partially prestressed beams after cracking. For further research data see also Chapter 14 of the author's book Vol.1 [29].

8. Other Problems

Unfortunately there is lack of space to deal with other problems such as composite sections, differential shrinkage, creep and stress redistribution, shear, torsion, compression or indeterminate structures. With regard to impact and economy see the author's respective papers (30) & (31), the latter presented in 1948.

Finally the author would like to acknowledge the facilities offerred him in preparing this paper by the Dept. of Civil Eng. of the University of Kentucky.

9. Conclusions

1. Members Class II with hardly visible cracks, have proved very satisfactory, provided that strict supervision and/or non-destructive, random performance tests are carried out. Otherwise there is a danger of great variation which might result in wide hair cracks and large deflection.
2 Class III ought to be subdivided into IIIA and IIIB, the former being in compression under "normal" service load with temporary visible cracks at the limit state of service load, and Class IIIB with permanently visible cracks.
3. Class IIIA represents the most suitable solution, when great differences

between limit state and "normal" service load occur, such as with highway bridges.

4. It is unnecessary with Class III, except for fatigue, to know the stresses under service loads, as the members must be designed for collapse load. It is only necessary to determine the required effective prestressing force.
5. This force must be large enough to compensate with IIIA the maximum tensile stress under "normal" service load and with IIIB the difference between the nominal tensile stress under the limit state of service load and the allowable nominal tensile stress. The latter indicates limitation of the strain and thus of crack width, as obtainable from tests.
6. Rigidity after cracking is governed by percentage of steel and bond efficiency. Non-tensioned mild steel ensures maximum rigidity, but is less economical and requires more space; prestressing strands are more economical, require less space, but rigidity is reduced. Lower strength strands may be preferable.
7. Non-bonded tendons cause few wide cracks, and ultimate resistance is limited. The Southampton tests have confirmed that these disadvantages are overcome by provision of well-bonded, non-tensioned steel. This allows restressing and avoids pressure grouting, but needs corrosion protection of the tendons.
8. Crack widths and deflection increase at sustained and fatigue loading, dependent on age and magnitude of stress at loading. Further research is necessary.
9. Further research of large range fatigue loading for a limited number of cycles is particularly important to obtain a basis for assessing the safe carrying capacity and the expected fatigue life of existing bridges and for future design in view of increase in "abnormal" loading. Such tests have been introduced at DUKE University which it is hoped will be continued.

References.

(1) IABSE, Final Report, 3rd Congress, Liege 1948;
(2) P.W.Abeles: "Safety against Cracking", IABSE, Final Report, 5th Congress, Lisbon, 1956, p.541;
(3) B.Thürlimann:"Report IVb", IABSE Prelim.Report, 8th Congress New York,1968;
(4) P.W.Abeles: "The Limit States of Reinforced and Prestressed Concrete", The Consulting Engineer, London, June 1968;
(5) F.Leonhardt:"Report IVa", IABSE Prelim.Report, 8th Congress New York, 1968;
(6) P.W.Abeles: "An Introduction to Prestressed Concrete", Vol.2, Concrete Publ. Ltd., London, 1966;
(7) P.W.Abeles: "Partially Prestressed Concrete Constructions at Eastern Region, British Railways", IABSE, Vol.12, 1952;
(8) P.W.Abeles: "The Conditions of Partially Prestressed Concrete Structures after 3-7 years' Use", IABSE, Congress Lisbon, 1956, p.625;
(9) T.Y.Lin: "Load Balancing Method for Design and Analysis of Prestressed Concrete Structures", ACI Journal, June 1963;
(10) P.W.Abeles: "Cracking and Bond Resistance in High Strength Reinforced Concrete Beams", ACI Journal, Nov. 1966;
(11) P.W.Abeles: "Design of Partially Prestr.Concrete Beams", ACI J., Oct.1967;
(12) W. Zerna: "Partially Prestressed Concrete" (in German), Beton u.Stahlbeton, Vol.51, No.10, Dec. 1956;
(13) G.H.J.Kani: "Prestressed Concrete"(in German), K.Wittwer,Stuttgart, 1955;
(14) A.F.Shaikh, D.E.Branson: "Unfavourable and Favourable Effects of Non-tensioned Steel in Prestressed Concrete Beams (to be published);
(15) J.O.Woolums:"Study of Behaviour of Prestressed Concrete Beams", M.Sc. Thesis, University of Kentucky, 1967;
(16) E.W.Bennett, N.J.Dave: "An Experimental Basis for the Design Calculations of Beams with Limited Prestress" (to appear in Structural Engineer, London in 1969);
(17) N.H.Burns, D.M.Pierce: "Stresses and Behaviour of Prestressed Members with Unbonded Tendons", PCI Journal, Oct.1967;
(18) P.W.Abeles, E.I.Brown, J.O.Woods: "Preliminary Report on Static and Sustained Loading Tests on P.C.Beams", PCI J., Aug. 1968;

(19) P.W.Abeles: "Fatigue Tests on Partially Prestressed Concrete Members", IABSE, Final Report 4th Congress Cambridge, 1952;
(20) P.W.Abeles: "Fatigue Resistance of Prestressed Concrete Beams, IABSE, Final Report 5th Congress Lisbon, 1956;
(21) C.E.Ekberg, R.E.Walter, R.G.Slutter: "Fatigue Resistance of Prestressed Concrete Beams in Bending", ASCE, July 1957;
(22) P.W.Abeles, F.W.Barton: "Fatigue Tests on Damaged Prestressed Concrete Beams", RILEM Intern.Symposium, Mexico 1966;
(23) P.W.Abeles, F.W.Barton, E.I.Brown: "Fatigue Behaviour of Prestressed Concrete Beams",ACI Intern.Symposium on Bridge Design, 1967;
(24) P.W.Abeles, E.I.Brown, J.Slepetz: "Fatigue Resistance of Partially Prestr. Concr.Beams to Large Range Loading",IABSE,8th Congr. IV(b)6;
(25) R.F.Warner, C.L.Hulsbos: "Fatigue Properties of P.C.", PCI J.,Febr.1966;
(26) R.H.R.Tide, D.A.VanHorn: "A Statistical Study of Static and Fatigue Properties of High Strength Prestressing Strands", Progress Report No.2, Fritz Eng.Report 309.2, 1966;
(27) J.B.Hilms, C.E.Ekberg: "Statistical Analysis of Fatigue Characteristics of Under-reinforced Prestressed Concrete Flexural Members", Report 545, Eng.Exp.Station, University Iowa, Ames, 1966;
(28) W,J.Venuti: "A statistical Approach to the Analysis of Fatigue Failure of Prestressed Concrete Beams", ACI J., Nov.1965;
(29) P.W.Abeles: "An Introduction to Prestressed Concrete", Vol.1, Concrete Publ.Ltd., London, 1964;
(30) P.W.Abeles: "Impact Resistance of P.C.Masts", IABSE Publ.,Vol.17, 1957;
(31) P.W.Abeles: "The Economy of Prestressed Concrete", IABSE, Final Report, 3rd Congress, Liege 1948.

SUMMARY

Beams with temporary visible cracks and great ductility at the "limit state" of service load, but fully prestressed and rigid at "normal" service load,offer an ideal solution of a structure, including highway bridges, having temporary, instantaneous cracks at "abnormal" load, provided a safe overall fatigue resistance is ensured. Studies in this respect are made at DUKE University. A combination of non-bonded tendons (allowing re-stressing) with a new type of well bonded, non-tensioned strand has proved very satisfactory. Recent research on crack width and deflection at static, sustained and fatigue loading is discussed.

RÉSUMÉ

Poutres avec fissures visibles temporaires et grande ductilité à l'état limite de service normal mais entièrement précontraintes et ainsi rigides au poids normal, semble être la solution idéale d'une structure, y compris ponts ayant des fissures instantanées temporaires sous poids anormal, pourvu qu'une entière sécurité de résistance soit assurée. Etudes sous contract sont en cours à l'Université de Duke. Une combinaison de tendons non-adhésifs (permettant la recontrainte) avec un nouveau genre de torons bien adhésifs non tendus, s'est démontrée très satisfaisante. On discutera de récentes recherches sur la largeur et la dévation des fissures qui se produira sous des poids statiques, continus et prolonges.

ZUSAMMENFASSUNG

Balken mit temporären sichtbaren Rissen und grosser Verformbarkeit im Grenzzustand der Nutzlast, aber voll vorgespannt und daher starr bei "normaler" Nutzlast bieten eine ideale Lösung einer Konstruktion, auch für Strassenbrücken, in welchen Risse unter "abnormaler" Last entstehen, vorausgesetzt dass genügende Schwingungssicherheit besteht. Solche Studien werden an der DUKE Universität gemacht. Eine Kombination zwischen nichtvermörtelten Spanngliedern (mit Ermöglichung von Nachspannen) und einer neuen Type einer schlaffen Litze hat sich sehr bewährt. Versuchsergebnisse über Rissweite und Durchbiegung bei statischer, bleibender und schwingender Belastung werden besprochen.