Partial Prestressing — A Historical Overview

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The idea of prestressing concrete began to be discussed in the late nineteenth century at which time proposals were made by Jackson in the United States (1886) and Dōhring in Germany (1888). However, the early experiments were unsuccessful, largely because of a failure to appreciate the effect of the creep and shrinkage of the concrete on the prestressed steel, and because there was not then available a high strength steel with sufficient strain capacity to accommodate the nonelastic contraction of the concrete without excessive loss of stress.

It was not until 1925 that high strength steel strands were first used for prestressing by the Spaniard Eduard Torroja, in the tie members of the Tempul Aqueduct. Shortly thereafter, the application of high strength steel to prestressed concrete structures began to be pioneered by Eugène Freyssinet in a series of important structures in France.

The Origin of a Concept

During the period preceding World War II the main advantage of prestressing was considered to be the counteracting of dead load by permanent prestressing forces and the modification of the stresses in the concrete under live load so as to eliminate cracking and enable it to function as a homogeneous material. In 1939, however, a different emphasis appeared in a proposal by the Austrian H. von Emperger.[1]

Emperger made the suggestion that a small number of pretensioned high strength steel wires should be added to

Note: This article is based on a paper presented at the International Symposium on Nonlinearity and Continuity in Prestressed Concrete, University of Waterloo, Waterloo, Canada, July 4-6, 1983.

the ordinary medium strength bars in reinforced concrete. His objective was not to eliminate cracking, which he regarded as desirable because of the flexibility which it imparted to reinforced concrete, but to increase the allowable service load by reducing the effective stress in the reinforcement.

This appears to have been the first statement of the concept of partial prestressing. Emperger supported his proposal by the results of a series of tests in which up to 42 percent of the reinforcement was replaced by wires, although the prestress in the latter was very low.

The following year Paul W. Abeles (Fig. 1), who had been a pupil of Emperger, endorsed the suggestion in a paper entitled "Saving Reinforcement by Prestressing."[2]

"It is a simplification and improvement to tension only a part of the reinforcement, consisting of thin wires, and to abandon the idea of having a homogeneous building material . . . until the final stage."

Nevertheless, Abeles also recognized that, below a certain level of loading, the stress in the concrete would be entirely compressive, and the advantage of a homogeneous uncracked material would therefore also be obtained with the partial prestressing arrangement proposed.

"The compressive stresses in the concrete tensile zone and the unstretched reinforcement . . . are reduced to zero if only that part of the total load acts which corresponds to the ratio chosen for the prestressing."

Abeles made the further suggestion in his paper that the reinforcing bars might be replaced by high strength steel wires identical to those used for prestressing, thus providing the required ultimate resistance by a greatly reduced area of steel and at a lower overall cost, even allowing for the greater unit cost of the high strength steel. An alternative method was to prestress the entire high strength steel to a lower stress.

Partial prestressing, as thus envisaged by Emperger and Abeles, could be regarded as a method of exploiting high strength steel in reinforced concrete. Whereas in ordinary reinforced concrete the necessary ultimate load could be obtained with a reduced area of high strength steel, it could only be done at the cost of large deflections and cracks of undesirable width at the service load. However, partial prestressing seemed to offer a new way of using high strength steel which would overcome this impasse and improve the behavior at service load. Abeles was convinced, by his experience in testing spun concrete poles and reinforced concrete beams made with high strength steel and concrete, that it would be possible in partially prestressed concrete structures to keep the deflection and width of cracks within acceptable limits, provided the reinforcement was well distributed and bonded.

Synopsis

The concept of partial prestressing originated in a proposal of Emperger in 1939 which was developed by Abeles against considerable early opposition. Various partially prestressed structures were built in England from 1949 onwards and design rules were drawn up in the Codes of Practice.

Research on partially prestressed concrete commenced in a number of countries in the 1960s and the provisions for partial prestressing in the 1968 Swiss regulations marked the beginning of a rapid growth of interest and extensive application in that country. This experience, and the increasing amount of international discussion, should result in the more widespread exploitation of partial prestressing in other countries.

Fig. 1. Dr. Paul W. Abeles (1897-1977).

Early Tests and Development

The proposal of Abeles immediately attracted considerable criticism from a number of engineers engaged in the development of prestressed concrete at that time, and his paper was followed by a lengthy published correspondence with K. W. Mautner. It was argued that many of the advantages of prestressing would be lost, that there would be very severe cracking and that the economic advantages of the proposal were dubious or nonexistent.

Both sides of a controversy that was to last for many years were presented vigorously, but at that time prestressing technology was still in its infancy so that it was not possible to discuss the economic factors meaningfully, and there was also little experimental data. Abeles aptly concluded the correspondence with the words "It will depend on tests to prove whether my ideas are adequate."

During World War II there was only limited opportunity in England for such tests, or indeed for any sort of development of prestressed concrete apart from the use of the material in railway sleepers as a substitute for timber.[3] However, Abeles continued to advocate the principle of partial prestressing,[4] and carried out some small-scale tests including one of a partially prestressed beam reinforced by wires of which 40 percent were prestressed. These tests demonstrated the excellent recovery of deflection and closure of cracks of prestressed concrete after overloading while some tests of used railway sleepers as simply supported beams showed that these properties were still present after over 2 years in service.[3]

When post-war reconstruction began in the late 1940s Abeles had the opportunity to begin to apply his ideas in practice as a result of the need to reconstruct a considerable number of railway over-line bridges to give the increased clearance necessary for electrification, particularly where masonry arches had to be demolished. Since the construction depth of the new bridge decks had to be reduced to a minimum it was decided to adopt partially prestressed concrete, using a composite solid slab incorporating precast prestressed beams of inverted T section as illustrated in Figs. 2 and 3.

Abeles convinced British Railways that partial prestressing could be an economical method in rebuilding their bridges without jeopardizing the safety of the structures. Fig. 4 shows the erection of partially prestressed precast inverted T beams for the Gilroyd Bridge on the Manchester-Sheffield railroad line in 1949.

The first bridge decks were designed to permit a tensile stress of 3.45 N/mm^2 (500 psi) in the concrete at service load, in contrast with the normal "fully prestressed" structure of that time in which no tensile stress was allowed. Tests consistently showed that in this type of beam with well-distributed pretensioned wires cracks did not become visible until the tensile stress was about twice the above value. As a routine inspection procedure one beam out of each row was test loaded (Fig. 5) so as to develop a tensile stress of 5.2 to 5.5 N/mm^2 (750-800 psi), at which it was specified that no cracks were to be visible. In one instance the test load was sustained for a period of 30 days, during which the deflection increased by 65 percent but no cracking of the concrete occurred (Fig. 6).

Although it therefore appeared that cracking of the concrete should not occur at the levels of stress which were now being permitted, concern was still felt about the possible results of cracking due to an accidental overloading of the structure, when there would be a marked increase in the tensile stress in the pretensioned wires. Fatigue of the wires was a particular danger under these conditions and it was therefore decided to carry out a repeated loading test of a partially prestressed composite bridge deck slab which had previously been loaded sufficiently to cause flexural cracking.[5] The cracked slab was first subjected to one million cycles of a load at which the stress in the concrete before cracking of the slab would have

Fig. 2. Brick masonry arch bridge before reconstruction (Gorton-Manchester line).

Fig. 3. Masonry arch bridge after reconstruction using composite partially prestressed concrete deck for overhead electrification. Two-span structure at 31 ft (9.5 m) was built for British Railways in 1951 (Gorton-Manchester line).

Fig. 4. Erecting partially prestressed inverted T beams for the Gilroyd Bridge on the Manchester-Sheffield railroad line in 1949. Dr. Paul Abeles is on the right.

Fig. 5. Load test of partially prestressed concrete inverted T beam at precast prestressed concrete plant.

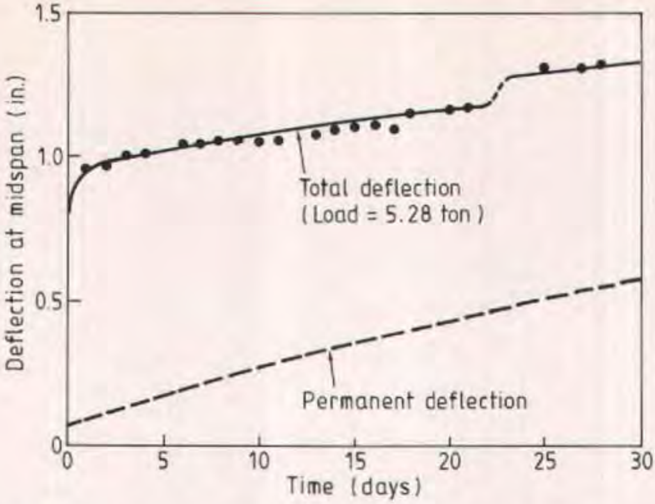

Fig. 6. Deflection test of partially prestressed concrete inverted T beam under sustained load. Deflection-time curves for two different allowable tensile stress conditions.

varied from 0.7 N/mm² (102 psi) compression to 3.8 N/mm² (553 psi) tension; for the second million cycles the maximum stress was increased to 5.5 N/mm² (800 psi) and for the third million cycles the range of stress was from 3.0 to 6.2 N/mm² (436-902 psi).

Although the opening and closing of the cracks with each repetition of load could be clearly observed, the cracks were still found to close completely so as to be invisible on removal of the load after the second million cycles, and after the third million cycles the cracks were only just visible. In a final static loading to failure the ultimate load was found to be about the same as that of a slab which had not been fatigued.

In order to develop the required ultimate moment the first group of partially prestressed composite bridge decks, the cross section of which is shown in Fig. 7, included mild steel nonprestressed reinforcement, placed in the in-situ concrete. Following a successful test[6] Abeles' original proposal was put into effect and high strength prestressing wires were used as nonprestressed reinforcement. The wires were placed together in pairs in the bottom flange of the precast prestressed beam as in Fig. 8, a development which both reduced the amount of site work and required only about one-fifth of the amount of nonprestressed mild steel reinforcement that would have been necessary, since under ultimate load conditions the stress developed in the wires had been shown to be almost equal to their tensile strength.

Partially prestressed beams with post-tensioned cables were first used for the roof of a freight depot at Bury St. Edmunds in 1952 (Fig. 9) and subsequently in a number of other roof structures (Fig. 10). Somewhat lower tensile stresses were specified than for pretensioning, but with successful experience and an increasing amount of test data the permissible tensile stresses were increased all round to 5.2 N/mm² (750 psi) for pretensioning and 4.5 N/mm² (650 psi) for post-tensioning. Stresses of 1.7 N/mm² (250 psi) were

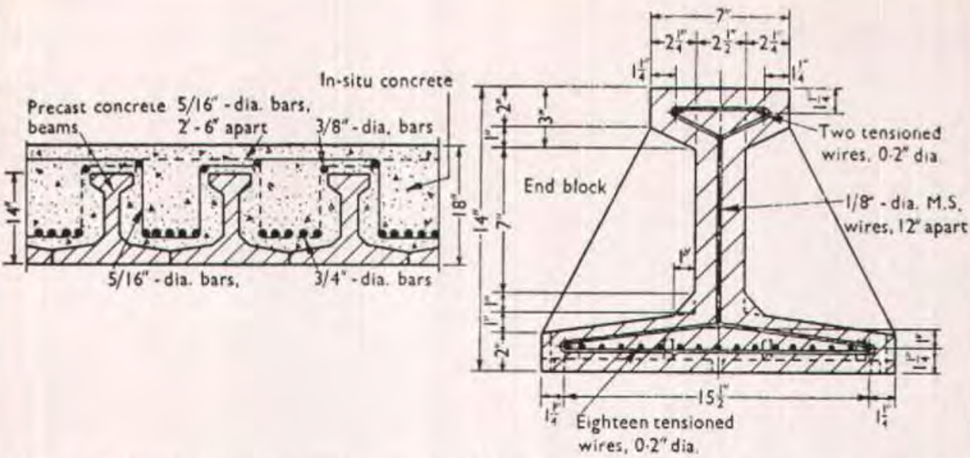

Fig. 7. Composite partially prestressed bridge deck with nonprestressed reinforcement.

used in railway under-line bridges.

In the development of a concrete mast for overhead electrification partial prestressing offered a particular advantage for uniformly prestressed sections subject to approximately equal moments in opposite senses. It was found that the reduced prestress resulted in a lower maximum compressive stress under load and in this type of prestressed section it was often the compressive stress that was critical. Moreover, in these members the severest effect was usually a temporary unbalanced load during erection or resulting from the accidental breaking of the overhead wire under which conditions high stresses were acceptable.

A further type of structure for which partial prestressing was found suitable was that designed to withstand mining subsidence, causing a severe loading not more than about once in the lifetime of the structure. Under these conditions severe cracking could be tolerated provided subsequent closure of the cracks and the absence of large permanent deflections could be assured. Examples of this application included jacking beams under the bearings of a steel girder bridge, a railway turntable foundation in the form of a wheel with prestressed rim and spokes, and the strengthening of a church tower by prestressing.[5]

The Partial Prestressing Controversy

By 1960 there had been considerable development and use of partially prestressed concrete in England, although almost entirely confined to the work of Abeles in association with the Eastern Region of British Railways, yet the controversy of 20 years before had been slow to die down. The early opposition of Freyssinet had been expressed on a number of occasions, particularly in a lecture at the Institution of Civil Engineers in London in 1949[7] in which he roundly declared:

". . . relative to a given state of load, a structure either is, or is not, prestressed. There is no half-way house between reinforced and prestressed concrete; any intermediate systems are equally bad as reinforced structures or as prestressed structures and are of no interest."

It is clear from the arguments given in

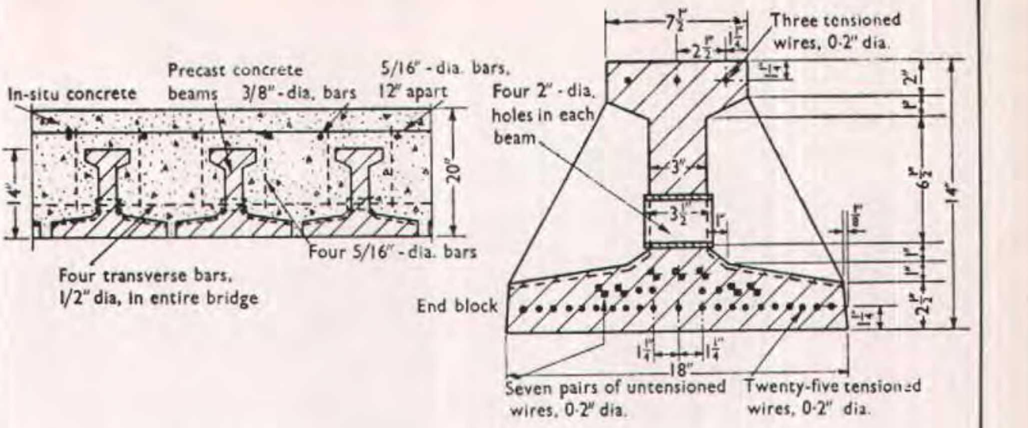

Fig. 8. Bridge deck with nonprestressed reinforcement of high strength steel wire.

support of this comment that Freyssinet had particularly in mind the behavior of a member such as a beam with prestressed reinforcement, possibly unbonded, at some distance from the tensile face ("not particularly well situated to function as reinforcement") and therefore "extremely sensitive to loads exceeding the transformation value." He spoke of the ratio of deflection to load being increased as much as ten times in the cracked, as compared with the uncracked, condition. This description, although correct for a certain type of prestressed member, is entirely inappropriate where there is well-distributed bonded reinforcement such as pretensioned wires or nonprestressed wires or bars of small diameter near the tensile face, as would be considered essential for a good partially prestressed design.

Freyssinet later modified his position and conceded that occasional tensile stresses of 5 N/mm² (727 psi) might and indeed should be permitted in a bridge,[8] but his achievements and prestige at this period were so great that statements such as the one quoted were bound to create early difficulties for the development of partial prestressing. Nevertheless, several engineers supported the concept, among them F. G. Thomas of the Building Research Station[9] and R. H. Evans[10] of the University of Leeds, who carried out tests of prestressed beams, some of them partially prestressed, as early as 1940.[11]

Early Design Recommendations

In 1951 the Institution of Structural Engineers published their First Report on Prestressed Concrete.[12] This was a very progressive early "state of the art" report, the authors of which were anxious not to limit the proper development of prestressing and adopted a very liberal attitude towards partial prestressing. Three types of prestressed structure were identified:

1. Structures in which the possibility of cracking at working loads should be avoided.
2. Structures in which cracks could be permitted under maximum live loads which occurred infrequently.
3. Structures in which visible hairline cracks could be permitted under

Fig. 9. Freight depot with partially prestressed concrete beams (Bury St. Edmonds, England, 1952).

frequently occurring working load.

The first type of structure was recommended for impact loading, under which no tensile stress was allowed and for fatigue loading, under which a tensile stress was permitted not exceeding half the flexural tensile strength of the concrete. For liquid retaining structures a load factor of 1.25 against cracking was specified.

In the second type of structure it was required that no tensile stresses should occur in the concrete under dead load alone, but with the live load acting the tensile stresses could exceed the flexural tensile strength, resulting in cracking.

In prestressed structures of the third type, design was based on the ultimate load alone and the stresses in the concrete at service loads were not required to be considered, provided there was no danger of fire, corrosion or fatigue.

The first British Standard Code of Practice for prestressed concrete appeared in 1959.[13] In this document the permissible tensile stress varied between 1.2 and 3.4 N/mm^2 (175 and 500 psi) depending on the duration or frequency of occurrence of the maximum loading, the strength of the concrete and the type of prestressing, whether pretensioning or post-tensioning. Under certain conditions this could be increased by up to 1.7 N/mm^2 (247 psi) and an unspecified higher calculated tensile stress was permitted where the maximum working load was "exceptionally high in comparison with the load normally carried," provided that under normal conditions the stress was compressive "to ensure that any cracks which might have occurred close up."

The above Code of Practice applied to the use of prestressed concrete in buildings, but the use of partial prestressing for bridge structures was not permitted by the British Ministry of Transport specifications, and under working load conditions even modest tensile stresses were excluded for many years.

Fig. 10. Partially prestressed roof beams for locomotive depot (Ipswich, England).

Widening Interest

During the 1950s most of the development of partial prestressing appears to have taken place in England, but the Fourth Congress of the FIP held in 1962 revealed activity in other countries.

In a report from the United States,[14] P. J. Verna described how difficulties had arisen because of the large upward deflection of precast prestressed flooring units in buildings where the permanent load was much less than the maximum load for which the units had been designed. This had been overcome by reducing the prestress so as to permit a tensile stress of $6\sqrt{f_c'}$ (stresses in psi) instead of zero stress under the full live load.

It was reported from Japan[15] that the introduction of partial prestressing for highway bridges had resulted in a 10 percent reduction of the initial prestressing force and a reduction of 3 percent in the total cost of superstructures. Theoretical and experimental studies had been carried out in Belgium[16] and the use of partial prestressing in the Soviet Union was mentioned in an oral contribution.

The years immediately following did not bring any outstanding developments, but there was a gradual increase of research activity. Abeles investigated the durability of cracked partially prestressed beams by overloading a number of specimens so that permanent cracks were formed; some beams were then exposed on the coast between the high and low water mark, while others were exposed to damp acidic conditions under the roof of a locomotive shed (Fig. 11). The high strength wires were in all cases found to be uncorroded on breaking up the beams after several years.

Abeles also supervised several projects in England as a research fellow at Southampton University and in the United States as a visiting professor at Duke University, North Carolina. Programs of tests were also carried out in England (University of Leeds and Building Research Station) and Switzerland (E. T. H. Zurich). Interest continued in Belgium where an international colloquium was held in 1965, and Brenneisen gave an invited lecture on partial prestressing at the sixth FIP Congress at Prague in 1970.

New Design Recommendations

A milestone was reached in 1968 with the introduction of partial prestressing into the new Swiss regulations (SIA—Norm 162).[17] Unlike the British First

Report and 1959 Code of Practice the limit of partial stressing was not defined by the tensile stress in the concrete, assuming cracking did not occur, but by the tensile stress in the prestressed and nonprestressed reinforcement, calculated for the cracked section. Under dead and live load the permissible stress in the nonprestressed reinforcement was 150 N/mm² (22,000 psi) and the increase in stress in the prestressed reinforcement was not to exceed one-tenth of the tensile strength, or one-twentieth in railway bridges. The steel stresses in the cracked section were less convenient to calculate than the concrete stresses in the uncracked section, but tables were produced for various sections and percentages of reinforcement to facilitate routine work in the design office. It was required that the concrete should be in compression when the structure supported only dead load.

A new code of practice appeared in Britain in 1972[18] which combined in one volume the design rules for reinforced and prestressed concrete, although it did not achieve an altogether unified treatment. Three classes of prestressed concrete structure were defined, following current CEB proposals. These were:

Class 1—Structures in which no tensile stress was permitted in the concrete under service load (i.e., "full prestressing")

Class 2—Structures in which a limited tensile stress was permitted, but in which there should be no visible cracking (this is sometimes termed "limited prestressing")

Class 3—Structures in which cracks of limited width ($\leq$0.2 mm) were permitted under service load (i.e., partial prestressing).

Calculations for Class 3 structures were to be based on the "hypothetical tensile stress" in the concrete, assuming the section not to be cracked. The permissible values of the hypothetical tensile stress varied according to the

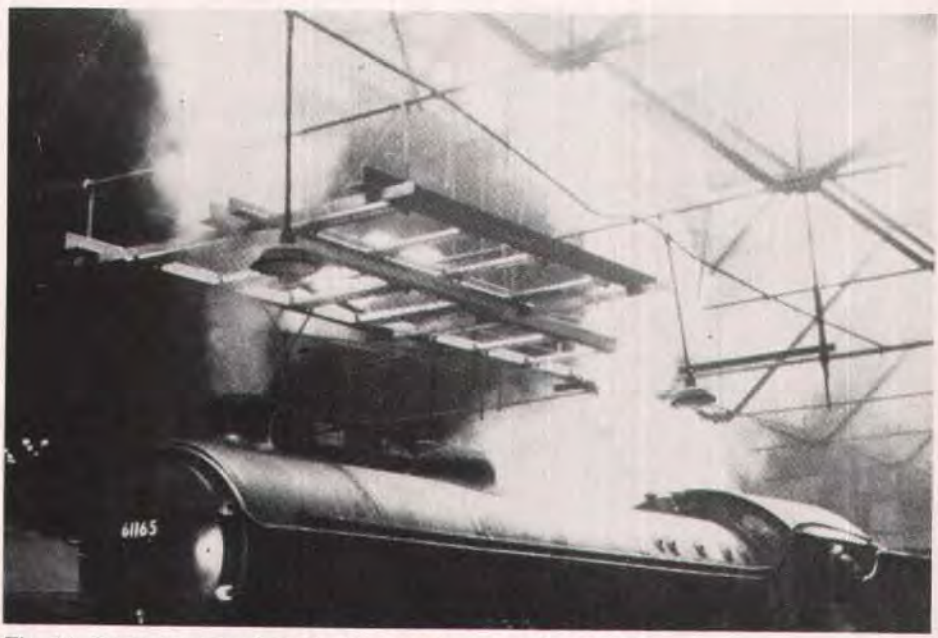

Fig. 11. Exposure test of cracked beams in locomotive shed.

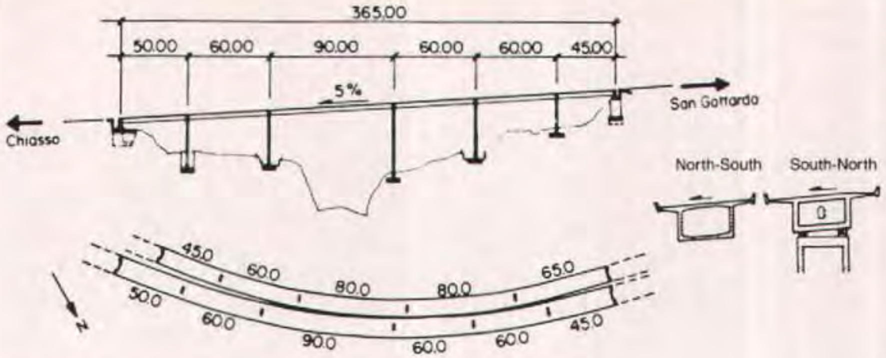

Fig. 12. Viadotto del Monte (St. Gotthard, Switzerland).

amount, type and distribution of the prestressed and nonprestressed reinforcement on the basis of empirical data provided by Abeles and others. From the viewpoint of accuracy the concept of the hypothetical tensile stress has shortcomings, but it offered the practical advantage that the method of calculation was similar for both Class 2 and Class 3.

The ACI Building Code (ACI 318-83) also uses the hypothetical tensile stress approach and permits a value of 12 $\sqrt{f'_c}$ (psi) with safeguards relating to minimum cover and check of deflection.

In 1979 an international inquiry conducted by the International Association for Bridge and Structural Engineering (IABSE)[19] revealed that eleven countries allowed limited prestressing in their codes but that only seven of these provided for partial prestressing allowing for cracking of the structure. Among them there was a considerable variation in the design provisions prescribed.

Progress in Switzerland

With the introduction of the above design provisions it was disappointing that British designers were slow to take advantage of the economies now possible with Classs 3 partial prestressing, and since its introduction there appears to have been relatively few applications, although Class 2 is now widely accepted. The same reluctance appears also to be found in the United States.

In Switzerland, on the other hand, the provisions for partial prestressing in the 1968 Code were adopted with enthusiasm over the next decade and by 1980 the majority of prestressed concrete structures were being designed in this way with highly satisfactory results, no cases being reported of damage attributable to partial prestressing. The structures built included major highway viaducts in the Alps and partial prestressing proved very advantageous for the transverse bending of the cantilevered top slab of box girders (Fig. 12).

In slabs the introduction of partial prestressing was found to result in considerable economy over reinforced concrete in the cost of reinforcement, and by column strip prestressing it was possible to reduce the peak moment and achieve crack and deflection limitation without introducing an uneconomically high degree of prestress at other parts of the slab.[20] Although the Swiss method assumed cracking at service load, and limited the stress in the reinforcement accordingly, in a larger number of structures thus designed the tensile stress in the concrete would not have been sufficient to cause cracking so that they could be considered to be in Class 2 according to the CEB definition.

The Next Step

The new Swiss design philosophy and favorable experience of its application caused a revival of interest in other European countries, notably Germany, the Netherlands and Belgium, in connection with defects which had arisen in some prestressed structures built in the 1960s according to the principles of full prestressing. Designed for the concrete to remain wholly in compression at all stages of loading, no provision had been made for cracking, the amount of non-prestressed reinforcement being minimal and not correctly placed for the control of cracks. Consequently, in certain instances where unforeseen overstressing of sections occurred, for example on account of settlement or temperature effects, the resulting cracks were widely spaced and of excessive width. Since the future prevention of such mishaps would require the provision of reinforcement it appeared logical to take advantage of the latter under normal service conditions by permitting a certain degree of cracking within the limits which had been established as safe through long experience with reinforced concrete.

Meanwhile Swiss engineers had reached the conclusion that, even under permanent load, well distributed hairline cracks could be accepted in prestressed structures as defined by the CEB Class 3. The Draft Code SIA 162 (1981) accordingly proposed a single requirement covering the stress in non-prestressed reinforcement in reinforced or partially prestressed concrete and the increase in stress in prestressed reinforcement in partially prestressed concrete. The permissible stress was related to the bar spacing close to the surface and varied from 280 to 90 N/mm^2 (41,000 to 13,500 psi) for spacings between 50 and 300 mm (2 to 11 in.). The ultimate moment, which had often been found to be the critical requirement, was to be checked, but no calculation of the stresses in the concrete was considered necessary except for fatigue conditions where full prestressing might still be necessary.[21,22]

Over the last decade Switzerland has undoubtedly led the world in the application of partial prestressing. The stage has now been reached in a number of other countries at which a considerable amount of research has been published and even international symposia have been held such as that organized by the FIP in Bucharest, Romania, in 1980[23] and at Waterloo, Canada, in 1983.[24] In England a report on partial prestressing has been published by the Concrete Society.[25]

Unfortunately, there has been no comparable activity in development and construction, without which research will eventually become sterile and unrealistic. It is hoped that one of the results of the June 1984 NATO Workshop in Paris will be a wider interest in the possibilities of partial prestressing which will lead to increased practical application and a determination to exploit and add to the knowledge and experience of more than 40 years.

Acknowledgment

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by May 1, 1985.

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