Contributions of Gustave Magnel to the Development of Prestressed Concrete

by L.R. Taerwe

Synopsis: In the early 1940’s Prof. G. Magnel performed extensive research programmes on real scale prestressed concrete beams at Ghent University (Belgium) in order to elaborate design methods for this new material. He also developed his own anchorage system which was used until the mid 60’s in Belgium. He gave many lectures in several countries in which he explained in a simple way the principles of prestressed concrete. He was also instrumental in the design of the first prestressed concrete bridge in the USA, the Walnut Lane Bridge in Philadelphia and he was the author of the first English textbook on prestressed concrete. He designed one of the first PC railway bridges in Europe and the first statically indeterminate PC bridge in the world. In the 1950’s many engineers from abroad spent some time in Magnels lab in Ghent to perform research and to get acquainted with practical realizations.

Keywords: history; Magnel; prestressed concrete
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THE "REINFORCED CONCRETE" PERIOD

Gustave Magnel (Fig. 1) was born in 1889 and graduated as civil engineer at Ghent University in 1912. From 1914 until 1919 he was employed by a London contractor and in 1919 he started his career at Ghent University. Due to his stay in London, he was not only fluent in French and Dutch (Flemish) but he was also very fluent in English which would turn out to be extremely useful for his later contacts in North-America. In those days, French was the main language used by engineers in Belgium and also a common language in international contacts within Europe.

In 1923 he published his first book "Pratique du calcul du béton armé" on the design of reinforced concrete. In the same year the Belgian Standards Institute published the first design guidelines for reinforced concrete structures, Magnel being the main contributor. One of his first technical papers dealt with the influence of column stiffness on the stresses in continuous reinforced concrete beams.

For Magnel it became clear that for the further development of reinforced concrete it was necessary to perform research, in other words, he needed a laboratory. After many political and financial difficulties he succeeded in founding the "Laboratory for Reinforced Concrete" in 1926, which was located in the basement of a former hotel. In this lab, Magnel had at his disposal a 300 kN universal testing machine and a 3000 kN compression testing machine. About the efforts he has to made to realise his laboratory and keep it operational he wrote the following : "The ultra-rapid evolution of technology, forces university institutes to adapt themselves continuously to the actual requirements at the risk of failing in their task. This adaptation can not happen on the initiative of the university management which, by definition, is not competent for it and, moreover, rather looks for savings than for new expenditures. Hence, it is the task of the professors to do the impossible to keep their teaching and research at the required level." He continued by stating the following : "It not only goes about having a laboratory : the question is to keep it operational, which requires additional funding. We obtain an extra income from testing we perform for contractors, companies and public authorities, ... ".

In 1937 the laboratory moved to a new building of the engineering faculty where much more space was available and some new testing machines were installed. By 1940, it had become the most advanced and sophisticated research and testing laboratory for reinforced concrete in the world. Magnel had little use for tests on small-scale models and in 1950 a special testing floor was installed, thus enhancing the facilities for loading tests on real scale reinforced and prestressed concrete elements. In 1975 the laboratory moved to a campus at the outskirts of Ghent where it is still located. In the 1990's the
name was modified into "Magnel Laboratory for Concrete Research", indicating that research on both material and structural aspects is performed.

**EARLY CONTRIBUTIONS TO THE DEVELOPMENT OF PRESTRESSED CONCRETE**

The first mentioning by Magnel of the principles of pretensioned concrete and post-tensioning by external cables was in 1940. These techniques were already used abroad and Magnel tried to convince Belgian companies to apply it as well.

During the second World War, Magnel was not allowed to teach. However, the Germans still allowed him to remain director of the lab he founded. During the secluded years at the laboratory, Magnel had the opportunity to conduct full-scale research on prestressed concrete girders. He also investigated on the phenomena of creep of high-strength steel wires and creep and shrinkage of concrete. During the war period, it was impossible for him to obtain the prestressing system developed by Freyssinet in France. Hence Magnel developed himself a post-tensioning system which became known as the "Belgian" or "Magnel-Blaton" system (Fig. 2). The anchorages of this system consist of several so-called "sandwich plates", arranged parallel to each other, and in contact with a cast-steel bearing plate. Each locking plate is provided with four wedge-shaped grooves in each of which two wires are secured with a steel wedge. In this way the stress in the different wires (typically Ø 5 mm) of one tendon is more uniform than in the case all the wires are stressed at once. Moreover, a fairly small jack could be used for stressing the wires (Fig. 3). The cable is placed in a sheet-metal sheath, or holes are formed in the concrete to permit the cable to be passed through the beam after concrete has hardened. Over the full length of the tendon, vertical and horizontal spacers were provided at regular distances which assured that the relative position of the wires remained the same along the tendon. Due to this arrangement there was a free space around each wire which allowed a good cover by the injection grout which is essential for protection against corrosion.

The Blaton-Magnel post-tensioning system was used in almost all prestressed concrete bridges in Belgium until the early 1960's. In these years it went out of use because it turned out to be quite laborious and strands were introduced to provide higher capacity tendons.

In 1946 he published his design method for statically determinate beams. He formulated the following four stress conditions under service loads in the critical section:

1. tensile stress at top fibre under initial prestressing and dead weight smaller than allowable tensile stress.
2. compressive stress at top fibre under prestressing and full load smaller than allowable compressive stress.
3. compressive stress at bottom fibre under initial prestressing and dead weight smaller than allowable compressive stress.
4. tensile stress under long-term prestressing and full load smaller than allowable tensile stress.
These inequalities were drawn as straight lines in a diagram with axes $1/P_i$ ($P_i$ : initial prestressing force) and $e$ (eccentricity) and resulting in a kern showing admissible combinations of $P_i$ and $e$ (Fig. 4). In principle, the eccentricity giving the smallest value of $P_i$ was chosen.

For shear design he took into account the beneficial effect of the prestressing force on the total shear force. He then looked for the section and the fibre where the principal tensile stress was maximum and compared this stress with an allowable value.

For the design of the end-blocks he proposed to calculate the shear force and bending moment in horizontal sections, which is the so-called deep beam analogy. The resulting shear and normal stresses were combined with the stress components from other sources to calculate the principal tensile stress.

In 1951 Magnel stated the following: "In my opinion, for each beam two calculations have to be made : the first based on stresses using the elastic theory, the other on ultimate load. However, it seems to be impossible at present to make this latter calculation accurately because all known methods require the use of coefficients, the value of which we really ignore. This is mainly true when the failure occurs by crushing of the concrete. I recommend the design based on stresses as the fundamental one, but as it does not always give the same factor of safety against ultimate failure, an attempt must be made in each case to check whether this factor of safety is sufficiently high."

In 1946 he had already tested a partially prestressed beam for which the cracking load was lower than the full service load as opposed to the original concept of prestressed concrete in which no longitudinal tensile stresses were allowed under service conditions. Although Freyssinet was heavily opposed to the use of partially prestressed concrete, Magnel realized that it could offer some advantages.

In 1947 he developed a practical solution for the case of statically indeterminate post-tensioned beams. He introduced the concept of secondary bending moments $M_{P,\text{sec}}$ generated by the prestressing force $P$, and defined the equivalent eccentricity as

$$e_{\text{eq}} = e + M_{P,\text{sec}}/P$$  \hspace{1cm} (1)

In this way, he could use a similar diagram $1/P_i$, $e_{\text{eq}}$ as for statically determinate beams. The relationships between the actual (geometric) eccentricities and the equivalent ones were calculated on the basis of expressions for the secondary moments.

As he had gathered sufficient theoretical knowledge and practical experience about PC, he wrote his first book on the subject. It was first published in French in 1948 and was soon translated in English and Spanish.
In the railway bridge over the "Rue du Miroir" in the Brussels' north-south train connection, two of the six bridge decks were realized in prestressed concrete (Fig. 5). For a span of 20 m a total depth of 1.15 m proved to be sufficient while a solution in reinforced concrete would have required an increase in slab depth by 0.7 m. This bridge was considered a pilot project and the related research project was partly sponsored by the Belgian Fund for Scientific Research. Hence, in 1944 Belgium was one of the first countries with a prestressed concrete railway bridge. On this project Magnel concluded the following: "In our opinion, prestressed concrete is the building material of the future. Over 10 years, very few bridges will still be built in reinforced concrete because everybody will be convinced of the advantages of prestressing. The realization of the pilot project in the Rue du Miroir will have been the catalyst of this evolution and all who were involved in it can be proud of the results obtained."

The first prestressed road bridges in Belgium were built in Zammel (start in 1944 ; 12 m span) and in Eeklo (1945-1946 ; 20 m span).

In 1947-1948, a new textile factory was built for the UCO company (Union Cotonnière) in Ghent which had, in those days, the largest roof structure worldwide in prestressed concrete, covering a surface of about 35000 m$^2$ and which is still intact (Fig. 6). One hundred primary beams with a span of 20.5 m and 600 secondary beams with a span of 13.7 m were necessary. All these beams were precast at the site at a rate of 3 primary beams and 18 secondary beams per week which required a perfectly organised casting yard. Magnel writes about this achievement: "During the last 3 to 4 months, this project attracts numerous architects, engineers and contractors both from Belgium and from abroad. They want to qualify themselves in the field of prestressed concrete, firstly in our lab and secondly at the building site".

In the same period a hangar for planes at Melsbroek, the former Brussels airport, was erected. The roof was supported by 17 post-tensioned beams having a total depth of 2.9 m and a mass of 300 tons (Fig. 7). Similarly as in the previous project, the beams were cast on the ground and, after post-tensioning, they were lifted into their final position.

In 1949, the famous Sclayn bridge over the river Meuse was constructed (Fig. 8). This was the first continuous prestressed concrete bridge in the world (Figs. 9 and 10). With two spans of about 63 m each, the bridge was also the longest prestressed bridge in the world. Due to the variable depth of the girder, the external cable profile was almost straight except for the kink at the central support. At that section, the secondary moment due to prestressing was equal to about 68 % of the local bending moment due to dead weight.

**THE WALNUT LANE BRIDGE**

In 1946, Magnel visited for the first time the United States of America as an "advanced fellow" of the Belgian-American Educational Foundation, founded by Herbert Hoover in
1920. The trip was organized by the late Charles C. Zollman, a former student of Professor Magnel at Ghent University. Later he became Magnel's unofficial representative in the United States, responsible for the detailed arrangements of Magnel's several trips to this continent. Mr. Zollman's early consulting services for the design and construction of pretensioning plants throughout the United States, his activities in the field of precast concrete as well as his many contributions to the PCI, have identified him as a pioneer of this industry in North America. During his first visit to the United States, Magnel lectured on prestressed concrete at several places, a subject almost unknown at that time in that country. Magnel had the rare gift to explain complex theories and difficult problems in a simple way and thus captivated large audiences.

Two significant effects occurred during Magnel's first visit to America which had a direct bearing on the development of prestressed concrete in America and which culminated in the realization of Philadelphia's Walnut Lane Bridge, the first prestressed concrete bridge in the USA (Fig. 11). The first event was the fact that Magnel was introduced to the Preload Corporation of New York, which eventually became a sub-contractor for the construction of the Walnut Lane Bridge girders. The second event was the fact that Magnel asked Zollman to translate the French manuscript of his book on prestressed concrete into English. After a lot of efforts and difficulties the book was published in London in 1948. The 6000 copies of the first edition were promptly sold out. Eight thousand copies of the second revised and expanded edition were published in 1950 and a third further edition was published in early 1954. During those early years, Magnel's book was the practical tool to which engineering students and practicing engineers referred to for the design and analysis of prestressed concrete structures. The impact of this treatise, as well as many of Magnel's other publications, had on the prestressed concrete industry is indeed significant. In the early 1950's, T.Y. Lin spent one year in Magnel Laboratory and after his return to the United States he published his book on "Design of Prestressed Concrete Structures" (1955).

In the late 1940's Ch. Zollman, who had joined the Preload Corporation in the meantime, could convince the Bureau of Engineering of the City of Philadelphia to realize the superstructure of the Walnut Lane Bridge in prestressed concrete on the basis of a proposal elaborated by Magnel. The Preload Corporation was awarded the sub-contract to fabricate the girders in 1949. In October 1949 a loading test was performed on a 49 m long and 2 m deep test girder, identical to the girders forming the center span of the bridge (Fig. 12). This test demonstration attracted some 300 engineers from seventeen states and five countries who stood in the rain for the entire day to witness the event. The successful testing to destruction at the job site, far away from the comforts of a laboratory, was a significant achievement which instilled public confidence in prestressed concrete. Ch. Zollman formulated it as follows: "No single event was more instrumental in launching the prestressed and precast concrete industry in North-America than the construction of the Walnut Lane Bridge in Philadelphia in 1950. More than anything else however, it was the charisma, the dynamism and engineering talent displayed by the man who designed the Walnut Lane Bridge, namely Prof. Gustave Magnel of Belgium, that gave the impetus necessary for the acceptance and development of prestressed concrete in the United States."
Prior and during the execution of the bridge, some problems needed to be solved because Magnel required a "zero slump" concrete. According to American practice this was not possible to realize but Magnel had to approve all execution details. Finally, a practical solution was found but several girders showed honeycombs and other imperfections\(^6\) (Figs. 13 and 14). The controversy had gone far enough to be published in the influential "Engineering News Record" with the headline\(^7\) : "Americans make soup, not concrete, says Belgian professor".

In October 1950, Prof. Magnel was awarded the Frank P. Brown Medal from the Franklin Institute in Philadelphia for his exceptional contributions to the development of prestressed concrete.

During the service life of the Walnut Lane Memorial Bridge, some of its girders underwent major repairs and finally it was decided to replace the entire superstructure. Site preparations to remove the existing girders began in 1989. The reconstruction of the bridge and the realignment of its approaches in 1990 stirred up nearly as much interest and curiosity in the engineering and construction community as did the original structure in 1949. The new girders, based on a modified hybrid standard AASHTO Type V girder, were all manufactured in a PCI-certified plant instead of the girders being constructed and post-tensioned on site.

THE SEQUEL

In the early 1950's several famous American researchers visited Magnel and his Laboratory, among which T.Y. Lin, David P. Billington (Princeton University) and Robert N. Bruce (Tulane University, New Orleans).

Together with other European pioneers of prestressed concrete, Magnel founded in 1952 the "Fédération internationale de la Précontrainte", abbreviated as FIP, which became a successful international technical organization. In 1996 it merged with CEB (Comité Euro-International du Béton) into fib (International Federation for Structural Concrete).

Magnel not only authored more than 180 technical papers but also published an impressive series of books on structural analysis and on the design of reinforced and prestressed concrete structures. In all these text-books practical design methods were given which use was facilitated by numerous tables and graphs.

During his last years, Magnel devoted a lot of energy to the preliminary design of a high TV-tower, which was planned for the World Expo 1958 in Brussels. The planned tower in reinforced concrete would have had a diameter of 100 m at its base and a height of 500 m. On top of the concrete part came a steel mast with a height of 135 mm. In those days this would really have been a world wide attraction. However, there was a lot of technical and political controversy about this project and finally it was decided not to build it.

Gustave Magnel passed away quite suddenly on July 5, 1955. This sudden loss was received with great sadness in Belgium as well as abroad.
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In October 1956, an academic session was organized in Ghent for the commemoration of Gustave Magnel, where also several of his former colleagues and friends from abroad were present. At that occasion Prof. R. Evans from Leeds University mentioned the following: "... His concrete laboratory was recognized as one of the best in the world. Hundreds of members of staff and research workers from a large number of universities have had the pleasure and privilege of visiting this excellent laboratory. Magnel always warmly welcomed at his laboratory those who wished to improve their knowledge. .... His gift of friendly intercourse enriched us all by their genial and mellow qualities. Although he often had strong views on technical questions, he was by nature so generous that it was a pleasure even to disagree with him."

In order to continue the commemoration of Prof. Magnel's exceptional achievements at long term, the General Association of Engineers graduated from Ghent University (AIG), bestows the "Golden Medal Gustave Magnel" every fifth year on the designer of a structure which is deemed to be an important and remarkable application of reinforced or prestressed concrete. The first ten recipients are: N. Esquillan (1959), P. Bloklan (1963), F. Leonhardt (1968), U. Finsterwalder (1973), R. De Keyser (1979), H. Wittfoht (1984), R. Greisch (1988), O. Olsen (1994), M. Virlogeux (1990) and J. Schlaich (2004).

CONCLUSIONS

From the previous overview it is clear that Prof. Gustave Magnel was an exceptional personality both from an academic and a human point of view. The following achievements can be pointed out:
- Contributions to practical design methods for reinforced concrete.
- Development of his own prestressing system.
- Development of design methods for prestressed concrete and authorship of the first English text book on the subject.
- Involvement in the design and realization of the first continuous prestressed concrete bridge worldwide.
- Contribution to the realization of the Walnut Lane Memorial Bridge in Philadelphia, the first prestressed concrete bridge in the USA.

REFERENCES

(3) Magnel, G., 1951, "Continuity in Prestressed Concrete", In R.P. Andrew and P. Witt (Eds.), Prestressed Concrete Statically Indeterminate Structures, Cement and Concrete Association, pp. 77-86.

Figure 3 – Jack for the Blaton-Magnel system

Figure 4 – Allowable kern for combinations of $1/P_1$ and $e$
Figure 5 – Railway bridge over the “Rue du Miroir” in Brussels (1943-1944)

Figure 6 – Textile factory with post-tensioned roof beams (1947-1948)

Figure 7 – Heavy beams for plane hangar at Melsbroek airport
Figure 8 – Bridge at Sclayn over the river Meuse

Figure 9 – Bridge at Sclayn: lateral view

Figure 10 – Bridge at Sclayn: cross section
Figure 11 – Walnut Lane Bridge in Philadelphia

Figure 12 – Loading test on girder for the Walnut Lane Bridge
Figure 13 – Honeycombs in the first girder cast in 1949

Figure 14 – Longitudinal crack on the outer sloping face of the bottom flange (south facia girder)
Early Applications of Prestressed Concrete in the United Kingdom

by C. Burgoyne

Synopsis: The initial impetus to the application of prestressed concrete in the United Kingdom was heavily driven by the conditions imposed by the Second World War. Refugees from Germany, particularly Mautner and Abeles, brought skills and knowledge of tests carried out in France and Germany. The need to provide emergency structures, both for military purposes and to meet civilian needs, together with shortages of many materials, especially steel, led to the construction of beams on a scale, and at a speed, which would not have taken place in more normal times. The paper covers developments in the UK from the outbreak of the war in 1939 up to the early 1950s, when more normal times returned. This period of development is still not properly documented because security conditions prevented the publication of full details of the applications.

Keywords: history; prestressed concrete; United Kingdom
Burgoyne

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INTRODUCTION

Prestressed Concrete was not developed in Britain, but several early applications took place there. The Second World War had a major effect since it significantly affected both the need for structural elements and the supply of materials.

Freyssinet’s used jacking at the crown of arches at Rairéals-sur-Besbre (1911), Veurdre (1912) and Boutiron (1913 – Figure 1) to overcome the effects of concrete shrinkage and creep, and he conducted laboratory experiments which eventually led to his influential patent in 1928. Glanville’s more theoretical work on creep at the Building Research Station in England was taking place at the same time, but it was undoubtedly Freyssinet’s work that led directly to prestressed concrete. He recognised that high-strength concrete and high steel pre-strains were needed to leave some prestress after creep had taken place.

The first publication drawing the attention of British engineers to prestressed concrete was a paper by Gueritte in 1936, read before the Institution of Structural Engineers; this was a translation of a paper by Freyssinet, who attended the meeting. Gueritte had, as early as 1931, read a paper before the British Section of the Société des Ingénieurs Civils de France, describing the views of Freyssinet; this was only one year after Freyssinet had extended his patent for the use of prestressed concrete. The 1931 paper was, however, more concerned with the bridge at Plougastel, near Brest, and the novel centring system used there. It refers to the properties of concrete lying somewhere between those of a solid and a liquid (an implicit reference to viscoelasticity), but gives few details. It also talks of the benefits of providing confining reinforcement to enhance the strength of the section.

The company of L. G. Mouchel and Partners, who had built up a pre-eminent role in reinforced concrete as holders of the U.K. licences, set up the Prestressed Concrete Company as early as 1937. The company does not appear to have become fully active until the arrival in the UK of Dr. Karl Mautner in the summer of 1939. Mautner had been a director of Wayss & Freitag in the 1930s, and was also a professor at Aachen. He was of Jewish descent, but despite holding the Iron Cross as a result of his distinguished service in WWI, he was rounded up with other Jewish professionals and placed in Buchenwald concentration camp in 1938. At that time, however, it was possible to buy oneself out of the camps and with the help of the Mouchel Company in England, and probably also of the British Secret Services, he came to England. He brought with him details of Freyssinet’s work, with which, as licensee, Wayss and Freitag had been actively involved.
Gueritte published two further papers in the early 1940s. The first, in July 1940, describes the mass-production of precast, pretensioned beams; it also describes the design of emergency bridge beams and the design of a 3-span post-tensioned river bridge with a main span of 48 m, the central 15 m of which were formed of a simply-supported span. There was also a technical appendix written by Mautner. The second paper, published in 1941, gave extensive details of a test carried out at Southall on a beam 8.8 m long (Figure 2), and also described the test carried out about 5 years before, in Germany, on a longer beam designed by Freyssinet and tested by Mautner. The various developments sprang from this work.

In this paper locations are given using the Ordnance Survey Grid Coordinate system, which consists of two letters and 6 numbers, sufficient to define any location to within 100m. These coordinates, when entered into UK-based web mapping systems, will produce detailed maps of each location, at a variety of scales.

**WAR-TIME CONSTRUCTION**

**Monkton Farleigh Mine.** The limestone mines at Monkton Farleigh in Wiltshire (ST799656) had been worked in the 18th and 19th centuries to extract a seam of high quality freestone that was used to construct the city of Bath. The seam was about 5 m thick and typically 25 – 30 m below the surface. It had been excavated using the pillar and stall method, leaving quite sizable columns of stone to support the overlying lower-quality limestone that was not extracted (Figure 3). This mine, and others in the vicinity, were used as bomb-proof bunkers to store ammunition and other ordnance before it was required. They covered some 80 hectares and were capable of storing about 350,000 tonnes of bombs. Monkton Farleigh mine was connected to the main Great Western Railway by means of a narrow-gauge railway tunnel and also by an aerial ropeway.

The original pillars left by the miners were irregular, and not suited to mechanical handling, so it was decided to replace the pillars by regularly-spaced walls, 5 m apart, and to use prestressed concrete beams at 1.5 m centres. Mr Allan of the Ministry of Public Buildings and Works, and Lt-Col Withers of the Royal Engineers, decided in 1939 to investigate the use of prestressed concrete for these beams, which were designed in consultation with Mautner and successfully tested. They were designed for a central point load of 81.5 kN applied at the centre of the span, and were clearly intended to restrain loose blocks of stone falling from the roof rather than supporting the whole of the overburden, so they do not appear to have been loaded initially. Access to the mine was by inclined adits. Figure 4, which was not taken specifically to include the beams, shows two such girders. They were supported at their ends by steel rods hanging from brackets let into the walls; in the photograph these have been sprayed with concrete, presumably as corrosion or fire protection.

The scheme went ahead, with the intention that every third wall would be built, after which the intervening pillars would be removed and the intermediate walls completed. However, the demolition of the pillars took place much more quickly than the building of the walls, with the result that large areas of the ceiling were left unsupported and some of
the walls that had been completed were overloaded\textsuperscript{13}. A collapse occurred and the whole rebuilding scheme was cancelled, with the result that only a limited number of prestressed beams were installed. Nevertheless, this appears to be the first use of prestressed concrete in the UK.

The beams are described in Gueritte’s paper in 1940\textsuperscript{7}; although he does not mention the mine by name it is clear that these are the beams to which he refers. The paper also gives details of the anchorage blocks used in the stressing beds (Figure 5). With plans to fabricate such a large number of beams in a very short period of time, long-line fabrication was adopted. Nine parallel prestressing beds were made, 151 m long, in each of which 28 beams could be produced at the same time. The beams were complex in shape; the drawings show them with rectangular cross-section at the ends but I-beam section in the middle, with a web whose thickness varied along the length. 282 such forms would have been required if all beds were in use simultaneously. Shear links were provided and individual link spacings were specified, which today would be regarded as impractical, although it shows the over-riding importance attached to the optimisation of the steel. The tendons were stressed to about 1140 MPa so extensions of nearly a metre had to be accommodated at the jacking point. This was done by stressing pairs of wires using a yoke, with packing being installed once the desired extension had been reached. Once the concrete had cured sufficiently, the jacks were reinstalled, the packing removed, and the prestress released. If all 3000 beams had been built, each mould would have been used 12 times, and allowing for transfer of prestress at an age of seven days, with another week in between for setting up the reinforcement and concreting, it would have taken some 6 months of flat-out operation to complete the order.

There was relatively little to see on the surface, with a view to avoiding the attentions of the Luftwaffe, but the site was bombed in August 1940; it is thought that the bombers mistook the stressing bed for the (much longer) runway on a nearby airfield which was their intended target. Figure 6 was taken by the RAF to check the site camouflage; it was deemed to be unsatisfactory.

The mine is no longer used to store ammunition, but some parts have been taken over for commercial secure storage of documents and computer media. As a result, large parts are inaccessible and none is open to the public. Some of the beams are still in place, although their present condition is unknown. Grote gives a sketch of the present layout of the beams\textsuperscript{6}.

**Railway Ties** (Sleepers in UK parlance). Prestressed concrete railway sleepers were introduced in 1942\textsuperscript{14}. There had previously been experiments with reinforced concrete sleepers, as early as 1917 in Ireland, with others on the LNER main line in 1928, but these were generally unsatisfactory as the reinforced concrete crumbled when subject to frequent heavy loads\textsuperscript{15}. Wartime difficulties in obtaining suitable hardwood for the traditional timber sleepers led to further experiments in 1941 with reinforced concrete\textsuperscript{16}, and some were put onto a branch line near Derby. The tests were inconclusive, so the next year 100 of the sleepers were placed on the main LMS line near Watford; they survived for 10 days. At the same time, development work was under way on prestressed
concrete sleepers, designed by Mautner for production on an experimental long-line stressing bed at the Royal Signals Research Establishment (RSRE) at Malvern. The first of these sleepers was used in a trial in 1942 and they were so successful that all work ceased on reinforced concrete apart from occasional use in sidings. A purpose-built factory was constructed by Dowsett at Tallington in 1943 and continues in production to this day (Figure 7). It has been estimated that after 50 years of production there were some 35 million prestressed, pretensioned, monoblock sleepers in use in the UK alone.

The sleepers are designed for point loads applied by the rail, to be resisted by upward pressures from the ballast (Figure 8). If the ballast is in good condition, support is concentrated under the rails, but after settlement the ground support can be much more uniformly distributed. The sleepers are normally in hogging bending between the rails, and sagging bending under the rails; the top profile is altered so that the tendons remain straight but are in the appropriate position to resist the different moments. No other reinforcement is provided in the sleepers apart from a fixing to attach the rails.

Emergency Bridge Beams. During the early part of WWII there was a considerable need for bridge beams for emergency repair. Many of these beams were made of steel, but it is known that a number of prestressed concrete beams were also made. It is not known how many beams were made nor, to a large extent, is it known where they were used. Gueritte describes standard beams designed for spans of 4.6 m, 6.1 m, 9.1 m, 12.2 m and 15.2 m. The beams were to be used side-by-side, with timber decking. In some cases, the beams were to be tied together transversely by prestressed rods, which also served to attach the handrails (Figure 9). It is clear that savings in the amount of steel required were one of the primary advantages seen for these beams; it is also clear that these bridges were expected to be temporary and were seen as a means of restoring communication in the event of bomb damage. One surprising aspect is the preponderance of box beams in the smaller sizes; although they are more likely to be stable when placed, without any need for a transverse tie, the internal shutter must have been difficult to form and wasteful of resources.

In the discussion to a later paper, Chettoe describes the construction method, which is very similar to that used for the Monkton Farleigh beams. He mentions three fabrication yards, using beds between 180 m and 250 m long, each with three or four parallel lines, but he does not give named locations. Presumably these did not include the facility at Monkton Farleigh itself, but it is very likely that they were situated at other Government research establishments, such as RSRE where the sleepers were made. It is clear that a very large number of these beams were probably built, but there is no central record of where most of them were used. A paper published in the Institute of Civil Engineers in 1943 refers to two bridges built with these beams; these were clearly intended as permanent installations. Mention is also made of another that was planned.

Bridge over the LMS in Lancashire. This bridge was one of those described in the 1943 paper (Figure 10). It has now been removed, but was located at OS Grid reference SJ595948, where the A49 road crosses the West Coast Main Line in Newton-le-Willows. The bridge was unofficially known as the Bloodystone Bridge since nearby was a
monument to the murder by a returning crusader of his wife’s lover. The bridge was unusual in that the beams were skew to both the road and railway; the girders available from the emergency bridge stock had to fit onto the existing bridge abutments and were not the correct length for the skew span (Figure 11). The bridge was assessed in the early 1990s and a 7 tonne weight limit applied; the original cast-iron parapet beam and the reinforced concrete slab at the side were replaced in 1993. The prestressed girders were replaced in 1997; one of the principal reasons for its removal was the lack of shear reinforcement linking the beams to the top slab.

**Bridge in North Yorkshire.** The other bridge referred to in the 1943 paper is at Sinderby, where the Great North Road, now the A1, crossed the Ripon to Northallerton Railway (Figure 12). It is at OS Grid Reference SE335811. The road was dualled in the 1950s by adding two further carriageways to the west of the existing road; the old road over the bridge now forms a little-used private access and is not part of the public highway. The railway line has closed, and the void beneath both bridges has been filled-in with earth and concrete, but the bridge deck, including the prestressed beams, remains in position (Figure 13). The degree of support provided by the in-filling must be doubtful, and it certainly prevents both ventilation and inspection. According to Thomas, this bridge may have succeeded the LMS bridge above since it is said to have incorporated ties to stop the beams spreading, which had occurred on the other bridge. Despite having similar spans and prestress to the bridge at Newton-le-Willows, and also having no steel linking the beams to the slab, this bridge has never had a weight limit imposed. The long-term future of the bridge is uncertain since the road is soon to be upgraded to motorway standard and the bridge will cease to have any use.

**POST-WAR CONSTRUCTION**

After the end of World War II, Britain's economy was in a very poor state, with serious shortages of materials. Indeed, food rationing became worse after the end of hostilities, and as much production as possible was diverted for export to earn foreign currency. The back-log of maintenance that had built up meant that there was a heavy demand for steel; one of the principal drivers for the adoption of prestressed concrete was thus the recognition that prestressed concrete structures typically needed only about 25% of the amount of steel for the flexural reinforcement and a slightly reduced need for shear reinforcement because of the axial prestress that was induced.

The stock of emergency beams was used to rebuild bridges. It is claimed that they were so cheap that new construction was unable to compete. No central record exists of where these beams were used, but four locations were mentioned in 1949 in a brief note “An Exhibition of Prestressed Concrete”; Linford (Hampshire), Bury St Edmunds (Suffolk), Tilemill (Berkshire) and Newport Fronbridge (Pembrokeshire). However, it is probable that a very large number of these beams existed and they would have been too valuable an asset simply to throw away. Detailed locations of these bridges, and any others made with the same beam stock, together with photos and condition reports, are still sought.
Adam Viaduct (1946) near Wigan carries the LMS railway from Wigan to the South West, over the River Douglas, at SD571051. It has four spans of about 8.8m each, with sixteen I-beams in each span (Figures 14 and 15). It was prestressed with the Freyssinet system. The advantages were claimed to be the speed of erection and the fact that ballasted track could be used; the heavier beams meant that the bridge was also less susceptible to vibration. This bridge is still in use and has recently (1999) been listed by English Heritage, a process that recognises its architectural or historical significance, and gives some degree of protection.

Nunn's Bridge, at Fishtoft near Boston in Lincolnshire, was the first application of post-tensioning in the UK, in 1948. The bridge has five beams 21.3m long, 1.1m deep below an integral top slab (Figure 16). Each beam has twelve Freyssinet 12-wire draped tendons. The bridge replaced a three-span brick arch built by Rennie which was temporarily retained to support the falsework. The bridge is still in use and shows no sign of rust staining or deterioration of the concrete; the only maintenance undertaken appears to be attention to the handrails. The bridge is located at TF367415.

Masonry Repair (1948). The first application of the Magnel system in the UK was to the strengthening of a church tower at Silverdale near Newcastle-under-Lyme in Staffordshire. The tower was suffering seriously from subsidence caused by coal mining. The badly-cracked masonry was injected with grout to consolidate the brickwork to form “beams” of appropriate shape. A channel was cut into these beams to receive the tendons which were stressed against steel anchorage plates. Subsequently, the chase was filled with concrete to protect the tendons. The intention was that, if any further subsidence occurred, the internal beams would behave as if simply supported and prevent any further cracking of the masonry.

Airport Taxiway (1949). A prestressed concrete taxiway was built at the London Airport at Heathrow. This was similar to a complete runway slab built by Freyssinet at Orly; the slab was only prestressed transversely, but was made up of a series of 45° triangles (Figure 19). Vertical rollers were inserted in the joints, so as the slab contracted transversely it pushed against end abutments, thus inducing a longitudinal prestress as well. This system neatly overcomes the friction that would arise if direct application of the longitudinal prestress were attempted. The slab was 120' wide by 355' long. At about the same time a prestressed concrete road was constructed at Crawley, but this was prestressed longitudinally, which must have caused significant friction to develop between the slab and the ground.

Prestressed concrete in buildings (1949). Two notable applications of prestressing to buildings took place in 1949. The first was the fabrication of roof beams for the new Heathcote Factory at Tiverton in Devon, while the second was a much larger application to a new storage and distribution facility for H.M. Stationery Office at Sighthill in Edinburgh. This was the forerunner of precast, prestressed building construction. The secondary roof beams were pretensioned in a factory, while the main beams and the secondary beams for the floor were precast on site and post-tensioned before lifting them into position. These beams used the Magnel system, and some of the tendons were curved up at the ends. This building is regarded as significant.
since it took the ideas of precasting and prestressing from the bridge community to the building community.

**Partially Prestressed Concrete.** Paul Abeles was another refugee who came to England just before WW2. He was a believer in what is now called partially prestressed concrete, where additional untensioned reinforcement was included in the beam. The idea was to increase the ultimate moment capacity in the beam, with the primary effect of the prestressing being to reduce the crack widths but not to prevent their formation. He published a paper\textsuperscript{31} espousing his views in 1940, which was severely criticised by Mautner in the subsequent discussion. The first major application of his techniques is believed to be the reconstruction of railway bridges for the electrification of the LNER railway out of Liverpool Street Station in London. The bridges had to be raised to allow clearance for the overhead wires, without needing to raise the road. These beams were discussed in a paper in 1952\textsuperscript{32}. His systems were openly criticised - there were many who said that rather than combining the advantages of reinforced and prestressed concrete it combined their disadvantages instead. That debate continues to this day!

**Conflicting systems.** In the years after WWII there were many conflicting prestressing systems. The Freyssinet system, under the guidance of Mautner and later Alan Harris, were already established in the UK and held valid patents, but immediately after the war Prof Magnel of Ghent University was actively publicising his system, and Abeles was proposing the use of partially prestressed concrete. Many other systems were developed; some to avoid the patents held by the early protagonists, but others with genuine improvements\textsuperscript{33}. It was during this period that today’s standard systems were developed using 7-wire strands held by 3-piece wedges.

**The history after 1950** is well-documented - *Concrete, The Structural Engineer* and the *Proceedings of the Inst. Civil Engineers* all carry many papers relating to innovations or landmark prestressed concrete structures, and there have been a number of papers and reviews giving details of prestressed concrete over the next 50 years\textsuperscript{34,35,36,37,38,39}.

**CONCLUSIONS**

While it is clear that prestressed concrete would probably have become a successful material anyway, the special circumstances that came together in the United Kingdom between 1938 and 1950 led to the more rapid introduction of the technique. Refugees from Germany brought skills and knowledge with them; pressures to produce a large number of beams very quickly for wartime purposes, combined with shortages of materials, especially steel which was needed for many other purposes, all contributed to the rise of prestressed concrete. Only after the war did questions about patents and commercial exploitation arise.

It is probably unsurprising that wartime security concerns meant that details of the use of prestressed concrete were not published more widely, and other technical achievements in the war, such as Radar, the Spitfire, codebreaking and more visible civil engineering
achievements, such as the Mulberry Harbours, have captured public attention. But the more effective use of concrete by prestressing also played its small part.

ACKNOWLEDGEMENTS

Figures 3, 4 and 6 were provided by Nick McCamley; Figure 11 by St Helens Council; Figures 14 and 15 by wiganworld.co.uk.

REFERENCES

7. Gueritte, T.J., Recent developments of pre-stressed concrete construction with resulting economy in the use of steel, (with a technical appendix by Mautner), *The Structural Engineer*, 18, 626-642, July 1940.
10. www.multimap.co.uk
13. Whitehouse J., letter to Director of Fortifications and Works, 14 May 1940
Figure 1 – Boutiron Bridge

Figure 2 - Southall beam test⁶
Figure 3 - Pillar and stall mining at Monkton Farleigh

Figure 4 – Two beams in place at Monkton Farleigh
Figure 5 - Anchorage block at Monkton Farleigh stressing bed

Figure 6 - Aerial view of the prestressing bed at Monkton Farleigh (ST797658)
Figure 7 - Original long-line prestressing beds for railway sleepers\textsuperscript{13}.

Figure 8 - Sleeper design process\textsuperscript{13}
Figure 9 - Emergency bridge cross sections (Redrawn from Reference 7)

Figure 10 - Plan of Bloodystone Bridge

Figure 11 - Underside of Bloodystone Bridge showing cast-iron edge beam and tapering reinforced concrete side slab.
Figure 12 - Cross section of Sinderby Bridge

Figure 13 - The only exposed part of the prestressed beams at Sinderby (below parapet)

Figure 14 - Adam Viaduct, Wigan
Figure 15 - Underside of girders at Adam Viaduct

Figure 16 - Nunn’s Bridge, Fishtoft

Figure 17 - Long section through church wall
Figure 18 - Plan of Silverdale church tower showing strengthened areas (Redrawn from Reference 24)

Figure 19 - Heathrow Airport Taxiway (Redrawn from Reference 28)
A Historical Review of Prestressed Concrete through Patents

by A. Schokker

**Synopsis:** Prestressed concrete is a relatively young form of construction in the United States. The development of the various types of prestressing materials, including anchorages was critical to the success of prestressed concrete as we know it today. This paper reviews the history of prestressed concrete with a focus on post-tensioning related patents in both the United States and abroad. Early patents from 1945 and onward are included up through the present day. The 1950’s and 1960’s are a primary focus as the decades where prestressed concrete was taking a foothold in the United States.

**Keywords:** history; patent; post-tensioned; prestressed
INTRODUCTION

The last 50 years have seen a relative flurry of activity in the area of prestressed concrete, and particularly in post-tensioned concrete. A look at related patents gives an interesting summary of the history of post-tensioned concrete. The patents referenced in this paper are from the library of Morris Schupack who was kind enough to donate them to me. These files included correspondence detailing thoughts on various patents. Any mistakes in the reporting of these patents are mine alone. The paper does not cover every patent in the area of post-tensioning, but rather a sampling of patents from these files with a focus on patents during the early years of post-tensioning in the United States.

RESEARCH SIGNIFICANCE

1940’s and 1950’s

Patents related to post-tensioning exist prior to the 1940’s, including the 1928 patent by Eugène Freyssinet of the “Manufacturing Process for Reinforced Concrete Components.” This patent is typically considered the birth of prestress, although the term itself was not in use until years later. Another major patent from Freyssinet is “Tensioning and Anchoring of Cables in Concrete or Similar Structures” in 1945. This early anchoring system is shown in Figure 1. Freyssinet’s anchorage for a bundle of single wires utilizes a cast-in cone (similar to today’s trumpet behind the anchor plate) with the wires fanning out around the cone and held in place by an interior cone. Wedges are located between the wires outside the perimeter of the inner cone. A “coil” of reinforcement is included around the outer cone. The system includes details about jacking to reduce anchor seating type losses. The patent mentions that the system could be used for pretensioning or post-tensioning, but the intent is use in a post-tensioned system with a steel duct and a grease or wax coating on the wires. The files indicate that the original application for this patent was in 1940 in France and then in 1941 in the United States, with approval in 1945.

Several patents are of interest from the 1950’s. Figure 2 shows a drawing from a French patent from 1951 (Dyckerhoff & Widmann) on the procedure for constructing a cantilevered post-tensioned bridge, including some basic tendon layout. Figure 3 shows an early patent for PT (post-tensioned) duct by Upson in 1954. This patent discusses the use of a flexible metal tubing that can be used in place of the practice of removable forms to form voids in the concrete for the prestressing wires. Several variations in corrugation are shown. The system is grouted and the duct shown appears to be very similar to much...
of today’s duct. However, the patent intends for the grouted duct to provide the anchorage of the wire with no permanent PT anchorage.

Fritz Leonhardt’s 1956 patent, “Apparatus for Anchorage of Concrete Reinforcements,” describes a system that utilizes separate wedges in an anchor body. The figures in the patent (one is shown in Figure 4) show a bundle of three wires within a single wedge, but the patent text includes an option for individual wedges for each wire or rod. The patent also includes a stressing system with a jack that power-seats the wedges. Daugert’s 1959 patent on “Wedge Anchors” illustrates a thin, tapered wedge with teeth similar to today’s strand wedges, but Daugert’s wedge was intended for use on PT bars. Other anchoring patents from the 1950’s include additional types of wedge systems (Stresssteel, 1956) and button-head wire (or “nodule”) systems (Brandt, 1959) as well as rod and bar anchoring systems (Finsterwalder, 1956; Wollcock, 1956; Lee, 1958).

1960’s

A large number of patents were approved in the 1960’s with a continued focus on anchoring devices, in addition to patents for stressing jacks and Prestressed reinforcement production. Correspondence found in the files with the patents included discussion about whether the wedge systems were preferred instead of the button-head system. New patents with improvements and changes to the button-head system continued to appear during these years. These include a 1965 patent by Kourkene for a button-head wire anchorage system with shims and a radially slotted anchor plate as shown in Figure 5. An expansive grout is used to hold the anchor plates in place. A recessed button-head system was also patented in 1965 by Martter. Other button-head type systems patented during the 1960’s include a flat tendon system by Middendorf (1967) and a coupling device by Howlett (1967). A large number of patents with button-head systems came out during the 1960’s with minor variance between them. The correspondence during this time reflects the concern that conflicts were likely to arise between companies due to the many seemingly overlapping patents.

Patents related to PT bars included a 1966 system from Middendorf for flat bars to be used as slab tendons. The bars included a deformed end, similar to the button-head anchor approach. Finsterwalder had much earlier patents on threaded bar with a nut anchorage, but the earliest patent that I had access to that showed a system similar to the PT bar systems used today was his patent from 1964, “Anchoring Means for Reinforcing Insets in Concrete.” Interestingly, the correspondence included a discussion on the difficulty of finding a manufacturer for this product in the U.S. and the likely market for these bars.

While the button-head system continued to be used, patents were also being approved for a number of wedge type systems. At this point, 7-wire strand systems were also showing up in many of the patents. Rice patented a monostrand anchoring device for 7-wire strand in 1966 (Figure 6). This patent included a pocketing system similar to what was commonly being used by a number of companies at the time, causing concerns about patent infringement with the then-current practice. The validity of the patent was
eventually questioned in a lawsuit of Atlas vs. Rice during the 1970’s. Additional worries occurred over patent infringement for systems for stressing multiple wires or strands simultaneously. Freyssinet’s 1945 patent for multiple strand stressing seems to have been overlooked by the patent office in the consideration of the newer patents. The 1945 patent may not appear similar to the newer multi-strand stressing patents, but the concept is very similar to the broad claims made in some of these patents. Other items, such as the inclusion of a standard grout tube also show up in claims in patents of this era even though this type of grout tube had been used in practice since the 1950’s.

Other patents during the 1960’s include a hydraulic jack that is capable of restressing the tendon after initial stressing (Paul, 1963) and an anchor system that includes a spiral type of anchor zone reinforcement (Kelly, 1968). La Marr patented a stressing system in 1966 for PT beam members that stresses two strands from opposite directions simultaneously as shown in Figure 7. This is similar to the procedure often used in stressing strand for Prestressed tanks. Quite a few less-than-practical patents for PT anchor systems also show up during this decade.

1970’s

By the 1970’s, a large number of patents for PT systems were showing up with little variation and similar to systems already in use. Some of the less-typical systems from this time included a patent by Howlett for “Multiple Tendon Anchorage” in 1970. This system used various divided anchor head configurations used as the wedging system as shown in Figure 8. Spring assemblies similar to pretensioning chucks show up for post-tensioning anchorage is several patents, such as Kelly’s 1970 patent illustrated in Figure 9. This patent also shows a 3 part, rather than a 2 part, wedge. Other multi-strand anchor systems from this time include Herbschleb (1974), Brandestini (1974), Welbergen (1975), and Davison (1975). A number of jacking systems for multi-strand tendons also show up in the 1970’s, including patents by Andrews (1974), Shorter (1974), and Surribas (1974).

In the area of slab tendons (both monostrand and two strand systems), patents show increasing concern for sealing the tendon. Examples include Stinton (1972), Huber (1972), Kelly (1972), Edwards (1973), and Brandestini (1976). A production method for greased and sheathed strand was patented in 1974 by Middleton. The first patent for PT grout came in 1973 by Schupack, entitled “Tendon Grouting Means.” The patent describes a grout with a gelling agent to reduce bleed. A similar type of water retentive grout is still used today.

SUMMARY

A review of the patents during the early decades of post-tensioning in the United States reveals an active time in the industry. Many iterations were needed to approach the systems that are standard today (and that continue to improve). This paper provided a glimpse at several of the patents from this era, including the associated patent art.
ACKNOWLEDGMENTS

This paper was made possible by a donation from Morris Schupack of a large number of post-tensioning patent files. I appreciate his generosity and this chance to share this history of post-tensioning.

REFERENCES

Patents referenced in the document are arranged below in order of date. All patents are U.S. patents unless otherwise noted.

1928 Manufacturing Process for Reinforced Concrete Components, E. Freyssient, 1928

1945 Tensioning and Anchoring of Cables in Concrete or Similar Structures, E. Freyssient, #2371882, 3/20/45

1951 Procédé de Construction de Ponts á Longue Portée en Béton Armé, Dyckerhoff, Widmann, French Patent #1039305

1954 Prestressed Concrete Structure, M.M. Upson, #2,677,957, 5/11/54

1956 Post-stressed Reinforcing Rod Anchor, Ulrich Finsterwalder, #2755657, 7/24/56

1956 Means for Tensioning Rods and the Like, S.R. Wollcock, #2761649, 9/4/56

1956 Apparatus for Anchorage of Concrete Reinforcements, F. Leonhardt, #2763464, 9/18/56

1956 Wedge Anchors, Stressteel Corp., Patent #218602, 9/20/56


1959 Post-tensioned Anchor Device, F.H. Brandt, assignor to Prestressing, Inc., #2867884, 1/13/59

1959 Wedge Anchors, Peter C. Daugert, #2916785, 12/15/59

1963 Hydraulic Jack for the Prestressing of Concrete Reinforcements, O. Paul, #3090598, 5/21/63

1964 Anchoring Means for Reinforcing Insets in Concrete, Ulrich Finsterwalder, #3119203, 1/28/64
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1965  Post Tensioning Concrete Reinforcing Wires, J.P. Kourkene, #3225499, 12/28/65
1965  Prestressed Tendon Anchor Means, R.P. Martter, #3225500, 12/28/65
1966  Method and Mean for Prestressing Concrete, K.H. Middendorf, #3255558, 6/14/66
1966  Apparatus for Post-Tensioning Concrete Structures, R.R. LaMarr et al, #3285569, 11/15/66
1966  Anchorage for Post-stressed Concrete Stressing Tendons, E.K. Rice, #3293811, 12/27/66
1967  Post-tensioned Prestressed Concrete Members, K.H. Middendorf, #3300921, 1/31/67
1967  Prestressing Methods, J.W. Howlett, #327380, 6/27/67
1968  Apparatus for Post-Tensioning Prestressed Concrete, W.F. Kelly, #3399437, 9/3/68
1970  Multiple Tendon Anchorage, J.W. Howlett et al, #3522582, 8/4/70
1970  Anchor for Post-tensioning Prestressed Concrete, W.F. Kelly, #3534228, 8/18/70
1972  Anchorage System for Stressing Concrete, Hugh Dana Huber and Jack C. Edgren, assignee Conenco Intn’l Ltd., #3685934, 8/22/72
1972  Anchor for Post-tensioning Prestressed Concrete, William F. Kelly, assignee Kelly Systems, Inc., #3703748, 11/28/72
1973  Anchorage Assembly for Prestressing Cables, Hugh Jeremy Willis Edwards, #3757390, 9/11/73
1973  Tendon Grouting Means, Morris Schupack, #3762, 937, 10/2/73
1974  Method and Apparatus for Tensioning and Anchoring Tensioning Members, James S. Andrews, assignee American Stress Wire Corp., #3787957
1974  Post-stressing of Reinforced Concrete Structures, Robert William Shorter, assignee Pre-Stress Pioneers Ltd., #3795949, 3/12/74
1974  Anchoring Device for Wire Strands in Prestressed Concrete Structures, Antonio Brandestini et al, #3820832, 6/28/74
1974  Method of Anchoring a Bundle of Reinforcing Wires or Strands for Pre-Stressed Concrete and Anchoring Construction, Jan Frederik Herbschleb, Albert Komijn, Hans Egvert Westenberg, assignee I.B.I.S. Nederland N.V., #3822422, 7/9/74

1974  Apparatus and Methods for Tensioning Cables, Jorge Juan Ventura Surribas and Juan Coll Morell, #3844023, 10/29/74

1974  Method of manufacturing a Sheathed Cable for Use in Post-Tensioning Concrete Structures, Thomas E. Middleton, assignee pre-Stress Concrete, Inc., #3849221, 11/19/74

1975  Apparatus for Anchoring Wires or Stranded Wires, Gerard Welbergen and Hans Rudolf Seigwart, assignee Bureau BBR Ltd., #3863302, 2/4/75


1976  Anchorage Body for Anchoring Tendons with Wedges, Antonio Brandestini, Hans-Rudolf Siegwart and Gerard Welbergen, assignee Antonio Brandestini, #3956797, 5/18/76
Fig. 1 – Patent Art from Freyssinet, 1945, U.S. Patent #2,371,882
Fig. 2 – Patent Art from Dyckerhoff & Widmann, 1951, French Patent #1,039,305
Fig. 3 – Patent Art from Upson, 1954, U.S. Patent #2,677,957
Fig. 4 – Patent Art from Leonhardt, 1956, U.S. Patent #2,763,464
Fig. 5 – Patent Art from Kourkene, 1965, U.S. Patent #3,225,499
Fig. 6 – Patent Art from Rice, 1966, U.S. Patent #3,293,811
Fig. 7 – Patent Art from LaMarr, 1966, U.S. Patent #3,285,569
Fig. 8 – Patent Art from Howlett, 1970, U.S. Patent #3,522,582
Fig. 9 – Patent Art from Kelly, 1970, U.S. Patent #3,524,228
They Wrote the Book on Prestressed Concrete

by W.N. Marianos, Jr.

Synopsis: The growth of prestressed concrete development in the United States was greatly influenced by the publications available to practicing engineers. A number of the early books and publications on prestressed concrete are reviewed and discussed in this paper. Pioneers of prestressed concrete development are viewed through their publications and the role they played in promoting prestressing. Two key conferences (in 1951 and 1957) are also considered. Figures include samples of important passages from the texts considered. Through all of these, some of the flavor and personalities of this important period can be experienced.

Keywords: concrete; history; prestressing
W.N. Marianos, Jr. was born in New Orleans, Louisiana. He received his B.S. in Civil Engineering from Tulane University in 1981, an M.S. in Structural Engineering from the University of California at Berkeley in 1985, and Ph.D. in Civil Engineering from Tulane University in 1992. Marianos is a Fellow of ACI and past chairman of joint ASCE-ACI Committee 423 on Prestressed Concrete. He is a consulting bridge engineer and a student of engineering history.

INTRODUCTION

Engineers—especially structural engineers—are a notoriously conservative group. One joke notes that you can always identify the structural engineer, since he (or she) will be wearing both a belt and suspenders. Researchers and pioneering designers can introduce a new technology, such as prestressed concrete. Wide-spread acceptance of new materials and technologies in civil engineering, however, is often dependent on development of specifications and manuals for use of the innovation. Without written guidelines, many engineers are reluctant to apply cutting-edge technology in their designs.

This paper surveys the early development of prestressed concrete texts. These were the references used by practicing engineers to apply the innovative methods of prestressed concrete to their structural designs. These texts offer a glimpse of the perception and reality of the early days of prestressed concrete.

Two references, while not a focus of this paper, are particularly useful in exploring the history of prestressed concrete. The first is the article “Historical Perspective on Prestressed Concrete” by David Billington, published in the Sept.-Oct. 1976 issue of the Journal of the Prestressed Concrete Institute (Billington, 1976). The second is the book Reflections on the Beginnings of Prestressed Concrete in America, published by the Prestressed Concrete Institute in 1981 (Prestressed Concrete Institute, 1981). This publication is a compilation of a series of articles celebrating the 25th anniversary of the Institute. These references provide a helpful historical context in which to place the texts discussed.

While much of the early development of prestressed concrete occurred in Europe, this paper will focus on its growth in the United States. All of the texts considered are in English (though several are translations from other languages).

THREE KEY EVENTS

The first prestressed concrete girder bridge in the United States was completed in 1950. The Walnut Lane Bridge in Philadelphia was recognized at the time as a significant innovation, and spurred interest in prestressed concrete in the United States.
In August 1951, the Massachusetts Institute of Technology hosted the First United States Conference on Prestressed Concrete. This event drew participants from a variety of backgrounds, including academia, engineering practice, construction, and manufacturing. The attendance of over 600 reportedly doubled the organizers’ expectations. (Dean, 1955)

The 1957 World Conference on Prestressed Concrete was another key event during this period. Some 1200 hundred delegates from around the world traveled to San Francisco for the conference. (Kelly, et. al., 1957)

This paper will focus on developments and publications up to and including the 1957 Conference Proceedings.

EUGENE FREYSSINET

Freyssinet was the engineering visionary behind prestressed concrete. He developed the concept in France during the 1920s, and worked unceasingly to bring it to fruition. Freyssinet viewed prestressed concrete as a totally new material, unlike any other developed to that time. While he wrote prolifically, few of his works were available in English. This limited his direct impact on the early prestressed concrete developments in the United States.

In 1936, Freyssinet wrote, “the author considers himself entitled to state that he has succeeded in creating a theory and the means of giving it practical application which class the combination of steel and concrete when treated in accordance with these new methods as an entirely new material possessing properties very different from those of ordinary reinforced concrete.” (quoted in Huggins, 1981) Years later, T.Y. Lin opined that “Freysinnet claimed that prestressed concrete is supernatural and cannot be solved by regular mechanics.” (Lin, 2001, p. 108). While this may somewhat exaggerate Freyssinet’s views, it is clear that Freysinnet considered prestressed concrete as entirely unique. Guyon quotes Freysinnet as stating: “Prestressed concrete is in no way an improved reinforced concrete. It has no common frontier with reinforced concrete.” (Guyon, 1953)

Guyon continues to quote: “Whether from the theoretical point of view or from the practical the essential characteristic of prestressed concrete, that to which it owes its remarkable properties, is a certain predetermined value of compressive stress in the concrete, a value such that all possible stresses are brought within elastic limits. There is no prestress worthy of the name unless this value is attained and kept.” In other words, prestressed members should always remain in compression under service loads.

GUSTAVE MAGNEL

Magnel taught at the University of Ghent in Belgium. He had contact with a number of American students through an international exchange program between Belgium and the U.S. He spoke fluent English (as well as French and Flemish). He visited the United...
States on several occasions in the post-war period, traveling the country and lecturing on prestressed concrete.

Magnel was accompanied on his 1946 tour by a former student, Charles Zollman. At the end of his trip, Magnel showed Zollman the first chapter (in French) of his newest project—a textbook on prestressed concrete design. Zollman volunteered to translate the work into English, and convert all formulas and charts from the metric system. The completed work was rejected by major U.S. publishing houses for lack of a projected market. Prestressed Concrete was finally published in 1948 by Concrete Publications Limited of London. The initial edition of 6000 copies sold out rapidly, leading to issuance of a revised second edition in 1950 (Zollman, 1981a). Figure 1 shows the title page of the second edition.

During this period Zollman was Design Engineer with the Preload Corporation of New York, a firm specializing in construction of prestressed concrete tanks. Preload submitted a proposal for construction of the Walnut Lane Bridge in Philadelphia, and brought Magnel onto the project as chief designer (see Figure 4). The intense interest in this project among American engineers made Magnel the leading influence on early prestressed concrete development in the United States.

According to Tadius Gutt, “American engineers became intrigued with the possibilities of linear prestressed concrete as a result of the:

- Influence of Professor Gustave Magnel of Belgium.
- Construction of the Walnut Lane Bridge.
- Publication of technical articles in Engineering News Record, Civil Engineering, the ACI Journal and other periodicals.
- The use of prestressing for circular structures.” (Gutt, 1981)

David Billington wrote “The single most significant characteristic of Magnel was his ability to communicate as exemplified by his teachings and prolific writings—and more importantly to translate those ideas to the English speaking world.” (Billington, 1976). Figure 3 gives Magnel’s introduction to the concept of prestressed concrete.

PAUL ABELES

Paul Abeles was a colleague of Magnel’s. He began research on prestressed concrete in 1940 in Britain. His book, The Principles and Practice of Prestressed Concrete, was published in the U.S. in 1949, and includes a “Foreword to the American Edition” by David Steinman. From today’s perspective, Abeles’ book has the clearest presentation on prestressed concrete of the early European texts. Figure 2 shows a descriptive illustration from Abeles’ book.

Abeles was a pioneer in the field of “partially prestressed concrete.” Figure 5 shows Abeles’ comparison of fully and partially prestressed concrete. In advocating partial prestressing in some circumstances, Abeles was diametrically opposed to Freysinnet’s
views. He does, however, foreshadow the modern view of “structural concrete” as a continuum including conventionally reinforced, partially prestressed, and fully prestressed concrete.

THE 1951 UNITED STATES CONFERENCE ON PRESTRESSED CONCRETE

As discussed earlier, this conference showed the great level of interest in prestressed concrete in the U.S. Thirty-eight presentations were divided into three areas: applications, materials, and design and research. Several papers discussed the recently completed Walnut Lane Bridge.

Authors of these papers included Charles Zollman, Ross Bryan, Bill Dean, Chester Siess, Arthur R. Anderson, J.R. Libby, J.R. Janney and Blair Birdsall, among others. Many of these continued to be leaders in the development of prestressed concrete in the U.S.

The Proceedings of this Conference aided the growth of prestressed concrete by providing papers on a variety of applications, including tanks, pipe, bridges, buildings, piling, and pavements. While none of these papers provided authoritative design criteria, they did offer an overview of the fledgling industry for the unfamiliar.

THE BUREAU OF PUBLIC ROADS CRITERIA FOR PRESTRESSED CONCRETE BRIDGES

This document was highly influential in the development of prestressed concrete bridges. Its early publication implied an acceptance of the prestressing concept by the Bureau (a predecessor to the Federal Highway Administration). E.L. Erickson led the development of this publication, which appeared in pamphlet form in 1952. This first version was limited to post-tensioned concrete. Later revisions addressed prestressed concrete as well. (Erickson, 1957)

The 1954 edition of the Criteria was sold by the Bureau for 15 cents a copy. Concurrent work included development of standard drawings for post-tensioned deck girder bridges, and eventual publication of a variety of standards for prestressed concrete bridges. (Zollman, 1981b) Figure 6 gives an example of material considered during the development of the Criteria.

Bill Dean, in his address to the 1957 World Conference on Prestressed Concrete, gave the Criteria development effort high praise. He stated: “The Bureau has furthered the general development of prestressed concrete far beyond the boundaries of the United States through the weight of its well deserved international reputation for sound and progressive practices. In this it has done much to eradicate the impression of an ultra conservative, stick by the standards, type of public service that the taxpayers too often associate with governmental bureaus. In furthering the art and science of prestressed concrete, the Bureau of Public Roads has done honor to itself.” (Dean, 1957)
OTHER TEXTS

Several other books on the subject were also published during this period. Guyon’s *Prestressed Concrete* was published in English in 1953. This is a translation of the French edition of 1951. Guyon was a student of Freyssinet and follows his mentor’s views on prestressing. The 1953 volume covers only simply supported beams. The second volume, covering indeterminate structures, was published in 1960. Freyssinet himself wrote the foreword to Volume 1.

Kurt Billig’s *Prestressed Concrete* was published in Britain in 1952. Billig was a professor at the University of Hong Kong at the time and had written several prior works on concrete. He was particularly interested in pretensioned concrete. The text concludes with a draft code of practice for prestressed concrete. A.E. Komendant’s *Prestressed Concrete Structures* was published in the United States in 1952. Komendant was a consulting engineer, and in his preface states that the book is intended “for the use of men in the field as well as those engaged in research.” (Komendant, 1952).

It is unclear how accessible or popular these books were in the United States. It can be noted, however, that neither Billig’s nor Komendant’s works are mentioned significantly in historical surveys of prestressed concrete.

T.Y. LIN

T.Y. Lin was a professor at the University of California at Berkeley when he became interested in prestressed concrete after hearing several lectures on the subject in 1950. Lin viewed the defining issue with prestressed concrete as control of the structure by variation of prestressing force and location. (Lin, 2001, p. 111).

Lin wrote *Design of Prestressed Concrete Structures* while on a Fulbright Fellowship at the University of Ghent. He also participated in research there with Magnel. Lin’s work was the first “American” textbook on prestressed concrete, and became one of the most widely used in the United States. Lin recalled that his wife, Margaret, was his editorial assistant, and she informed him that “Every day you’ve got to write ten pages,” though this goal was not always achieved! (Lin, 2001, p.111).

Publication of *Design of Prestressed Concrete Structures* boosted both Lin’s prominence and the growth of prestressed concrete. The text format and nomenclature are familiar to U.S. engineers and it lacks the “foreignness” of several of the other publications discussed in this article.

T.Y. LIN AND NED BURNS

As T.Y. Lin tells the story: “Ned Burns was my co-author after the first edition. After the first edition went so well, John Wiley [the publisher] said, ‘It's time to revise it.’ So they got for me a very capable Ned Burns, who was a very good professor at Texas.” (Lin, 2001, p. 111). Ned Burns has remained the co-author on the book through subsequent
editions and revisions. *Design of Prestressed Concrete Structures* remains one of the most popular texts on prestressed concrete in the world, and has been translated into a number of languages. Its dedication remains as T.Y. Lin wrote in the first edition: “To engineers who, rather than blindly following the codes of practice, seek to apply the laws of nature.” (Lin and Burns, 1981)

THE 1957 WORLD CONFERENCE ON PRESTRESSED CONCRETE

Charles Zollman wrote that “for those who were directly involved (at times agonizingly so) … the late forties and early fifties were the golden age. Unhampered by stifling and restrictive building codes, which as yet did not have any provisions for prestressed concrete, they could design on the basis of their own design criteria and engineering judgment. They were not afraid to accept the responsibility for the structures they conceived and designed in the new material, even though they did not always fully understand the short term, much less the long term, behavior of prestressed concrete structures. But they were wise enough to be careful in their endeavors.” (Zollman, 1981b).

The 1957 World Conference on Prestressed Concrete provides a logical end to the “golden age” that Zollman describes. Figure 7 shows scenes from this conference. Bill Dean, Bridge Engineer for the State Road Department of Florida and a pioneer in prestressed concrete bridge development, gave the opening remarks, entitled “Prestressed Concrete—A Youth Comes of Age.” He discussed the 1951 United States Conference, and recalled prestressed concrete as a “challenging novelty which many of us approached in 1951 with an attitude of interested skepticism.” By 1957, however, it had “a firm and well deserved place in American practice.” He went on to describe the “largest program of public works ever conceived” – the Interstate Highway system and related highway construction. He predicted (accurately) that many of the thousands of bridges required would be prestressed concrete structures.

In conclusion, Dean stated: “Those who have been associated with the prestressed concrete field for the past several years have reason to be proud. Their past efforts have been spectacularly satisfying, present development is stimulating, the future appears to be not only promising but almost fantastic in its potential for the use and maturity of prestressed concrete design. For in the utilization of this economical, versatile, and highly adaptable material we are barely coming of age.” (Dean, 1957)

T.Y. Lin served as Chairman of the Conference, and gave its closing presentation “Looking Toward the Future.” In it he presented the parody shown in Figure 8. Lin closed his comments with: “In conclusion, I would like to quote ‘what is past is prologue.’ When it comes to Prestressed Concrete ‘you ain’t seen nothing yet!’” (Lin, 1957)
CONCLUSION

A historical review of the pioneers of prestressed concrete shows their unique personalities and approaches to their subject. A spirit of daring—at least by contemporary structural engineering standards—was essential to bring the potential of this innovative material to fruition.

In his introduction to Guyon’s *Prestressed Concrete*, Eugene Freyssinet touched on the importance of their non-technical attributes, writing: “…that if, among the certitudes which I have acquired during a half-century of construction and research, there is one which is abundantly certain, it is that virtues of character—courage, probity, respect of and love for the task accepted—are infinitely more necessary to the engineer than those of intelligence, which is never more than a tool in the hands of a moral being.” (Freyssinet, in Guyon, 1953)

Through the writing of these prestressed concrete pioneers and the writings of others about them, today’s designers can see a snapshot of that early “golden age.” This glimpse of the past can only increase the respect and appreciation due these bold and innovative engineers.

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REFERENCES


Gutt, T.J., 1981, “Prestressed Concrete Developments in the Western United States,” *Reflections on the Beginnings of Prestressed Concrete in America*, Prestressed Concrete Institute, Chicago.


Prestressed Concrete Institute, 1981, *Reflections on the Beginnings of Prestressed Concrete in America*, Chicago.

Zollman, C.C., 1981a, “Magnel’s Impact on the Advent of Prestressed Concrete,” *Reflections on the Beginnings of Prestressed Concrete in America*, Prestressed Concrete Institute, Chicago.

Figure 1 — Title page of Magnel’s *Prestressed Concrete* (Magnel, 1950).

Figure 2 — Figure from Abeles’ *The Principles and Practice of Prestressed Concrete* of Magnel’s illustration of prestressing (Abeles, 1949).
THE PRINCIPLE OF PRESTRESSED CONCRETE

Concrete has a compressive strength equal to many times its tensile strength. It is fairly easy to make concrete with a crushing strength of more than 6000 lb. per square inch on 6-in. cubes after 28 days, but it is more difficult to obtain tensile strengths higher than 350 lb. per square inch or bending resistances higher than 850 lb. per square inch at the same age. This is the reason why concrete is reinforced with steel bars placed in the direction in which tensile forces act. No account is taken of the tensile stresses developed in the concrete around the reinforcement, and in many members the tensile strength of the concrete is neglected. Under the working load the concrete on the tension side of a reinforced concrete beam is generally cracked since it is unable to conform to the normal strain in the steel.

The second weakness of ordinary reinforced concrete is that the dimensions of a beam are determined by the diagonal tension; if the shearing force is high a very large beam is required and for long beams the dead load becomes too great for practical purposes. Further, the shrinking of concrete during hardening may result in cracks even in the absence of load.

Finally, in ordinary reinforced concrete, full use cannot be made of high-strength concrete; that is, if the size of a beam were reduced beyond a certain limit, the amount of reinforcement required would make the beam uneconomical. It might be said that it would be sufficient to replace mild steel with steel having a higher yield stress so that the area of steel could be reduced to, say, one-sixth of its previous value; but this solution is not acceptable since the strain of the high-strength steel would be about six times the strain of the mild steel, and this would cause wide cracks in, for example, a beam under working load.

Prestressed concrete is a remedy for these weaknesses.

Figure 3 — Magnel’s introduction to the concept of prestressed concrete (Magnel, 1950).

Fig. 14. A very happy Professor Magnel conducted the test demonstration and delighted the spectators with his humorous running commentary.

Figure 4 — Magnel at the load testing of a prototype beam for the Walnut Lane Bridge (Zollmann, 1981a).
Figure 5 – Abeles’ comparison of full and partial prestressing (Abeles, 1949).
Figure 6 — Figure from Erickson’s article on development of Bureau of Public Roads bridge criteria (Erickson, 1957).
Figure 7 — Scenes from the 1957 World Conference on Prestressed Concrete (Kelly et. al., 1957).
The development of prestressed concrete can perhaps be best described by this parody, which Lin presented before the World Conference on Prestressed Concrete in San Francisco, 1987.

All the world's a stage,
And all engineering techniques merely players;
They have their exits and their entrances.
Prestressed concrete, like others, plays a part,
Its acts being seven ages. At first the infant,
Stressing and compressing in the inventor's arms.
Then the curious schoolboy, tenderly created
By imaginative engineers for wealthy customers;
Successfully built but costs a lot of dough. Then the lover,
Whose course never runs smooth, embraced by some,
Shunned by others, especially building officials. Now the soldier;
Produced en masse the world over, quick to fight
Against any material, not only in strength, but in economy as well.
Soon the justice—codes and specifications set up to abide,
Formulas and tables to help you decide. No more fun to the pioneers.
But so prestressed concrete plays its part. The sixth age
Shifts to refined research and yet bolder designs,
Undreamed of by predecessors and men in ivory towers.
Last scene of all, in common use and hence in oblivion,
Ends this eventful history of prestressed concrete
As one of engineering methods and materials
Like timber, like steel, like reinforced concrete.
Like everything else.

When Lin wrote the first edition of this book, prestressed concrete in the United States had barely entered its fourth stage—the beginning of mass production. Now it has emerged from the sixth stage and into the last. This rapid advancement has made possible a rather thorough revision of the previous two editions.

Figure 8 — Preface to third edition of *Design of Prestressed Concrete Structure* (Lin and Burns, 1981).
Figure 9 — Autographed page from the writer's copy of Design of Prestressed Concrete Structures (Lin and Burns, 1981).

by K.W. Kramer

Synopsis: A survey of the first uses of precast, prestressed or post-tensioned folded plates roofs show that this structural system start in the 1950’s and end in the 1970’s. A historic perspective of this structural system will be presented starting in the 1950’s and ending in the 1970’s. While prestressed concrete was making a break-through in the United States with the Walnut Lane Bridge in Philadelphia, PA, Europe was using prestressed concrete in various types of structural systems. One of these systems was the folded plate roof system. Approximately a decade later in the United States, the Cloverleaf Lanes Bowling Alley in Dade County, Florida used the same application in which the corrugated slab spanned 120 feet and extending transversely 286 feet. In 1962, this system of folded plate shells went from long span structures to being used in roof structures for dormitories at Washington State University. The main beams for the dormitories were pretensioned in a factory, while the secondary beams were post-tensioned on site. During the 1970’s, precast folded plate structures were being constructed throughout the United States for various types of buildings, one notable structure is the Hangar for Allegheny Airlines at Logan Airport, Boston, Massachusetts. Then they disappeared in the United States. Some Possible reasons why folded plate construction stopped are numerous but the two main factors are the architectural solution, and the lack of understanding of folded plate structures on the part of engineers and architects.

Keywords: folded plate; post-tensioned; precast; prestressed
INTRODUCTION

A folded plate is a type of shell structure. The Dictionary of Architecture & Construction defines a folded plate as “Construction consisting of thin, flat elements of concrete, steel, timber, which are connected rigidly at angles with each other (similar to accordion folds), forming a stiff cross section which is capable of carrying a load over a long span.” Figure 1 shows schematically a folded plate. Folded plates are essentially narrow beams which span longitudinally. Transversely, folded plates act as continuous slabs on elastic supports. Folded plates can be of various forms – the focus will be of V-shape and U-shaped plates. The terms “hipped plate,” “hip plate,” and “prismatic shell” were used in the United States prior to and in the 1950s. Sometime in the following decade the term “folded plate” was coined partially due to the ASCE committee on shell structures. The term “folded plate” is more nearly like the original German term, “Faltwerke,” for this type of structure.

In order to fully understand the evolution of prestressed/post-tensioned folded plate structures in the United States, a basic understanding of the history of cast-in-place folded plate structures, and of the precast/prestressing industry in the United States is required. Once this understanding exists, a more in depth examination of prestressed/post-tensioned folded plate structures in the United States can fully be appreciated including: an overview of the different analysis methods used; research and testing; and codes and standards.

BRIEF HISTORY OF CAST-IN-PLACE FOLDED PLATES

Folded plate construction was first used for coal bunkers erected in Germany in 1925. This type of construction was widely used in Europe and Russia on a great number of bunkers (Figure 2), roofs, and other structures during the 1920’s through the 1950’s. Although concrete folded plate structures were introduced to the United States, in the early part of the twentieth century their development and use prior to 1950 was infrequent. One possible reason is that the construction environment in Europe was prefect for shell structures. The critical shortage of building materials overseas during and following World War II encouraged experimentation in means of improving and extending the applications of available materials. Since metals were classified as strategic materials, concrete was the natural answer to almost all building needs. Another possible reason was the publicity given to the shells of Felix Candela, a Spanish architect and engineer living in Mexico. For a decade, starting from 1956, architectural magazines were full of examples of shell structures built by many different designers. This influenced architects to provide a modern building of unusual construction. Several variations of the concrete folded plate were popular – precast panels welded together
Although most folded plate structures in the United States were built in the 1960’s, one notable cast-in-place folded plate was constructed in 1936 consisting of a long span roof structure for a warehouse in San Francisco, CA designed by L. H. Nishkain. This roof structure covers a free floor area of approximately 45 feet by 70 feet (13.7 x 21.3 m) and is supported by four columns at the corners. The three plates forming the roof are 2.5-inch (6 cm) thick slabs supported on 4-inch by 8-inch (10 x 20 cm) joists spaced 2 feet (61 cm) on center, spanning only the 16 feet (4.9 m) of each of the three surfaces. The longitudinal members are merely shallow ribs rather than load-carrying members.

During the 1950’s, folded plate structures gained wide acceptance on account of their modern appearance which architects sought; and their superior performance and economy. By use of precasting, formwork costs were drastically minimized and construction, if plant casted, was carried out under well controlled conditions. It should be noted that several of the precast folded plate structures were cast on site and then lifted into place. In addition, prestressing was used advantageously to obtain crack-free concrete and to prevent early deterioration of the structure.

BREIF HISTORY OF PRECAST, PRESTRESSED CONCRETE

Prior to discussing prestressed/post-tensioned folded plate structures constructed in the United States during the late-1950’s to the early-1970’s, a brief look at the history of precast and prestressed concrete in the United States is presented. The use of prestressed concrete was introduced in the United States in combination with precast construction. Since the 1950’s, most projects using precast units have taken advantage of prestressing. Therefore, the discussion of precast concrete and prestressed concrete are collectively presented.

Although basic principles of prestressing were conceived in Europe, in 1886, P. H. Jackson of San Francisco, CA patented methods of tightening steel tie rods in precast concrete arch sections used in floors of buildings. Between the conception of prestressing in the end of the late 19th Century and 1949, a few precast/ prestressed concrete systems were patented in the United States. One system was developed by R. E. Dill of Alexandria, NE in 1925 to produce precast/prestressed concrete members such as posts and slabs. Dill used high-tensile steel, coated with a plastic substance to prevent bond, which was tensioned after the concrete had set. Subsequently in the United States to date the most notable precast, prestressed concrete structure is the Walnut Bridge which was in 1949 built in Philadelphia, PA.

During the same time period as noted above, Europeans had made considerable progress in the prestressing industry. For the same reasons as the development and use of folded plate structures, European designers evolved new prestressing techniques. One of their most important developments was solidifying the concept of prestressing into a workable, practical method of construction. Another factor which fostered prestressing in
Europe may be attributed to the different engineer-contractor relationship which existed there - one European firm assumes the responsibility for both the design and construction of a structure. Such an arrangement provided the financial incentive needed for the development of the concept of prestressing. Alternatively, in the United States, initial acceptance was slow because the efficient use of materials characteristic of prestressing could not be capitalized upon until means had been devised to reduce labor costs. Steps to minimize labor mainly took the form of developing production-line techniques for the prefabrication of prestressed members.

From 1949 to 1959, an astonishing increase in the use of precast, prestressed concrete occurred in the United States. Larger, heavy-duty truck cranes became available. Transit-mix trucks increased in size and efficiency, which undoubtedly increased the use of site fabrication of precast concrete. The advent of high strength steel, the 7-wire strand, and high strength light-weight and normal-weight concretes consisting of 5,000 psi and 6,000 psi (352 and 422kg/cm²) compressive strengths enabled prestressing to come to the forefront in the construction industry. In 1959 at the Prestressed Concrete Institute Convention, J. D. Piper presented a paper entitled *A Panorama of Prestressing in the United States* which states:

“One of the most perplexing phenomena on the American economic scene is the prestressed concrete industry – perplexing, that is, to economists. Never before had they experienced such a rapid acceptance of a major innovation in design and construction techniques. Upon a closer look, however, prestressing can be seen to have had a sound foundation of research, development and field trials before the first flush of popularity in the United States.”

Many revolutionary changes in the prestress/post-tension industry occurred during the late-1950’s to early-1970’s – load-balancing technique of analysis, full scale testing of prestressed folded plate structures with normal weight and light weight concrete, and development of codes and standards. Further discussion of these topics is included under the *Analysis Methods, Research and Testing, and Building Code, Standards, & Institutions* sections of this paper.

**PRESTRESSED/POST-TENSIONED FOLDED PLATES**

In the beginning of folded plate movement, it was not uncommon to have precast/ prestressing as an alternate on the construction documents of folded plate projects in the late-1950’s. Contractor’s then would work in close cooperation with an architect, structural engineer, and a precast/ prestressing concrete manufacturer to develop a folded plate roof unit designed for a specific job. After having success with the precast/ prestressed folded plate roof structure, the architect and structural engineer would specify on their next project, where appropriate, this system again. This was the beginning of prestressed/post-tensioned folded plate construction in the United States. This beginning appears to have started in California and Florida in the mid- to late-1950’s. By early-1960’s, folded plate construction had moved up the coast to Washington and across the United States. Prestressing proved beneficial in shell roof construction, especially folded plates as shown in Figure 3. As mentioned earlier, several variations of
the concrete folded plate were popular: prestressing/post-tensioning folded plates allowed engineers to use larger spans and provide architects with slimmer edge beams. Various projects will be given as examples of the different types of folded plate structures common during this time period. Typically, the projects are one of the first documented structures of a particular style using the method discussed. The Langendorf Bakery is used for the example of a cast-in-place prestressed folded plate structure. The second project is the Cloverleaf Bowling Lanes which uses cast-in-place, post-tensioned folded plate roof structure. A hangar for Mackey Airlines will be the example of prestressed precast panels used in a folded plate roof structure. Next, a circular dining hall at Washington State University utilizing various connections used for precast, prestressed panels will be discussed. Lastly, the Hangar for Allegheny Airlines at Logan Airport, Boston, Massachusetts will be presented as one of the most notable post-tensioned folded plate structures built in the United States.

Cast-in-Place Prestressed Folded Plate

In 1956, the largest example of cast-in-place, prestressed folded plate construction was the Langendorf Bakery in Los Angeles, CA. The plant area comprised 127,500 SF (11,845 SM). The roof of the production area which comprises 62,300 square feet (SF) (5,788 SM) is constructed of lightweight concrete folded plates. One of the unique aspects of this project is that little was known at the time of construction about prestressing of light-weight concrete and the loss of stress in the strands. The plates span 170 feet (51.8m) and are 4-inches (10 cm) thick except for extra thickening where prestressing anchorages occur. The structural design followed the principles of folded plate theory, as modified by the engineer, John Driskell, to account for plate deflections and the application of prestressing techniques. Straight prestressing cables were used for the following reasons: change of slope of cables was so large that excessive frictional losses would occur, resulting in widely varying resultant forces; taking account of friction losses in design, calculations leads to complicated mathematical notations and when the rather questionable distribution of such friction losses is considered the reliability of the final analysis is subject to great doubt; and to obtain the required prestress force in interior regions would have required additional prestressing material to make up for the friction losses. Therefore, it was decided by the engineer that straight cables in the inclined plane would be most economical, as friction losses would then be low.

At this time, almost any of the prestressing systems available in the United States could be employed in prestressing folded plate roof structures, provided there was some means of enclosing and protecting the required anchorages. Anchorages usually were located at the ends of the flexural members; and in some cases architectural design did not permit a concrete converge beyond this line. Due to architectural constraints, the Freyssinet system (Figures 4 and 5) for the prestressing was employed. The Freyssinet system permits the anchorage cones to be placed almost to the outside face of the building walls, as there were no permanent projections necessary to hold the stress in the wires. The building is still in use and is currently occupied by Escon Door Company.
Precast, Prestressed Concrete Folded Plates

The first application of prestressing a folded plate in Florida was initiated on the Mackey Airlines nose hangars at the Broward County International Airport in Fort Lauderdale (Figure 6). The structural engineer for the project was Walter C. Harry. A deep section, 3 feet 9 inches (1.1 m), was developed which was capable of providing main spans of 120 feet (36.6 m) and cantilevers 50 feet (15.3m). A cast-in-place stiffener was required at the supports and the structural design was simplified by the introduction of a series of ties between the top flanges above the floor. As a result, extensive transverse reinforcement was eliminated and heavy galvanized mesh was substituted. The thickness of the folded plate was increased to 3-1/2 inches (9 cm) for proper fire rating and ease of concrete placement. The weight of the unit was less than 60 pounds per square feet compared to a standard T-unit weighing 80 pounds per square feet. No special roofing was used on this project, the steepness of the roof, the quality of the concrete, and joint caulking provided adequate water-proofing. The hangar is still in use today.

Connections for Precast, Prestressed Folded Plates

A circular dining hall built in 1961 for Washington State University, shown nearly complete in Figure 7, is used to illustrate various connections for precast panels in folded plate construction. The structural engineer on the project was Halvard W. Birkeland. The center of the dining hall consists of a cast-in-place concrete core. The gable end beams at the outer wall of the dining hall are welded to the columns. The notch in the beam has an imbedded angle and the roof plate is welded to it. The roof plates are 43 feet (13.1m) long by 13 feet (4.0 m) wide by 4 inches (10 cm) thick and were cast in stacks adjacent to the building. After welding the joints are dry packed. The longitudinal radial joints of the plates are shown in Figure 8. The reinforcing bars are projected out of the plates into the joint and are joined by welding the flat bars to them. The circumferential joints and the connection to the support are shown in Figure 9. The plates were laid on a thin layer of grout. Continuity over the center support for live loads was obtained by the projecting bars, splice angles, and welds. The connections were completed at the center support by interlocking reinforcement and fill concrete; and they were welded at the end supports. The project is still being used today on the campus of Washington State University.

Cast-in-Place, Post-Tensioned Folded Plate

Cloverleaf Lanes Bowling Alley – At the beginning of the folded plate movement in 1958, the Cloverleaf Lanes Bowling Alley project in Dade County, Florida, shown in Figure 10, is an example of a cast-in-place folded plate concrete roof with post-tensioning. The structural engineer for the project was James P. McGlinchy, prestressing by R. H. Wright, contractor was John B. Orr, and the architect was Alfred Browning. The project required provisions for a 34,000 SF (3,159 SM) area – 50 bowling lanes – unobstructed by columns. The solution consisted of a folded plate spanning approximately 120 feet (36.6 m) and extending transversely 286 feet (87.2 m) for the length of the building. The overall depth is 6 feet (1.8 m); the horizontal module is 22 feet (6.7m) top
and bottom flanges are 8-inches (20cm) thick and the inclined webs are 5-inches (13cm) thick.  

Each interior corrugation of the Cloverleaf Bowling Alley is reinforced with 14 Roebling cables; 8 of these are placed straight in the bottom flange and 3 are placed parabolically in each web. All cables are composed of 12 wires, 0.276 inch (7 mm) in diameter. These wires are anchored in Freyssinet cones and grouted after tensioning. Figure 11 shows tensioning of the cables. Mild steel was used to provide for temperature stresses and load distribution in the transverse direction. Special mild steel reinforcing was added in the tip corrugation to provide for construction loads. The roof design followed the requirements of “The 1958 Tentative Recommendations for Prestressed Concrete of the ACI-ASCE Joint Committee 323”. Valuable design aids were ASCE Proceedings Paper 1580 “Design of Folded Plates” by Howard Simpson and Y.Guyon’s “Prestressed Concrete”. The latter was used as a basis for the end anchorage design. Refer to Figure 13 and 14.

The specified concrete strength of 5000 psi at 5 days with a slump of between 2 and 3-inches was obtained by means of Type III cement and a retarder. Double face forming was specified to insure that the concrete could be properly vibrated without displacing it down the slope as shown in Figure 12.

Allegheny Airline Hangar – At the end of the folded plate movement in 1970, the world’s longest free-span ever used for a folded plate roof structure was constructed. According to the chief structural engineer on the project, Sepp Firnkas, who investigated and researched similar structures, it also represented the smallest amount of materials used per square foot of covered floor area for any structure with comparable span (Figure 15). The Allegheny Airline Hangar footprint is 252 feet by 145 feet (77.00 x 44.20 m) with a structural depth of 14 feet for the roof structure which was the maximum allowed due to airport height restriction requirements. Several structural systems were evaluated: cantilever systems of 160 foot spans – steel tapered truss, concrete prefabricated sections post-tensioned and cast-in-place prestressed concrete folded plates; four post systems of 252 foot by 145 foot – steel truss system, steel plate girder system, steel space frame, concrete prestressed beams, and concrete folded plate. Guyed wire cantilevered systems were eliminated immediately because of height limitations. Truss and cantilevered systems showed large end deflections and would not fit into the available budget. Steel plate girders, prestressed concrete beams, and folded plate structures received close structural and economical scrutiny according to Sepp Frinkas.

The post-tensioned folded plate was the final choice with a total depth of 14 feet (4.27 m), inclined plates are 6 inches (15 cm) thick and the bottom plates are 12 inches (31 cm) thick and the top plates are 18 inches (46 cm) thick. The span to depth ratio of the structure excluded an ordinary reinforced concrete design. Therefore, prestressing was included in the design. The folded plate is supported at each end by a prestressed beam spanning 131 feet 5 inches (40.1 m) between columns. These beams rest on four rocker bearings which allow rotation of the roof structure in both directions and permit
transfer of wind loads; and creep, shrinkage, and temperature effects to the columns. A normal weight concrete with a compressive strength of 5,000 psi (352 kg/cm²), prestressing steel with ultimate strength of 270,000 psi (18,990 kg/cm²), and mild steel yield strength of 40,000 psi (1812 kg/cm²) were used. Partial stressing with an initial concrete strength of 3,200 psi (225 kg/cm²). To account for the effects of shrinkage, creep on stress losses, and long term deformations a “sustained modulus of elasticity” was obtained by multiplying the elastic modulus by one-half. This reduction factor was determined by taking into account the effects of strength, amount of cement paste, water-cement ratio, gradation and type of aggregate, and curing.Individual plates were considered as beams in the longitudinal direction. Shear strains were neglected since H/L ≤ 1/5. Bending moments of plates in longitudinal direction and corresponding shear forces and tensional moments were neglected. End diaphragm was assumed rigid in its plane and perfectly flexible normal to it. Material was assumed linearly elastic, isotropic and homogeneous (Hooke’s Law applied). Longitudinal strains and stresses were assumed to vary linearly over the depth of the plate. A computer program based on a multiple plate analysis was used. The actual geometry of the individual tendons was quite complex due to the three-dimensional configuration of the plates and had to be determined by a separate computer program. Plate thickness was determined by computer analysis based on allowable stresses and required post-tensioning forces. Computations were simplified by using the load-balancing concept.

Construction of several full scale mock-ups of a 30 foot section covering the area in which tendons drape from horizontal to the flange into a 45 degree angle on an inclined plate was done to study the reinforcing interferences, placement of concrete, and vibration. The final decision was to use a top form. On the recommendation of the prestressing supplier, the standard unit anchorage was changed to a wedge lock on a solid bar to allow greater flexibility in manufacturing and to accommodation tension length during placement as shown in Figure 17. The tendons were delivered at the job site in coils, then extended to full length and raised at the center by a wooden yoke. The tendons were then pulled towards the end diaphragm, anchored at one end, and set according to a drape previously laid out on the form surface as shown in Figure 18. Figure 19 shows concrete placement on the inclined folded plate. Filler strips between plates were cast after 25 percent of the prestressing force was applied. The hangar is still being used today.

ANALYSIS METHODS

Three methods of analysis were used for prestressed, folded plate structures: elastic, ultimate strength, and load-balancing.

Elastic Analysis

Similar to conventional structures, the design and analysis of prestressed concrete was first based on the elastic theory. A set of allowable stresses was prescribed, corresponding to the working loads. Concrete is visualized as being transformed by
prestressing into an elastic material and a family of formulas were derived taking the shape of $f = Mc/I$.

The first paper on the design of folded plate design theory was published in Germany by G. Ehlers and H. Craemer in 1930. The most notable design method used in the United States in beginning stages of folded plate structures was the Winter and Pei method which developed a distribution method in addition to solving a number of simultaneous linear equations with minor simplifications the stresses in the members were found. An extension of this theory to account for joint displacements was published in 1953 by Gasgar. This latter modification, however, had been applied in practice before the papers were published by those engineers engaged in this type of design.

The analysis of folded plates was a hot topic of numerous papers in the early 1960’s. A critical review of various design theories was presented by E. Traum in his paper entitled “The Design of Folded Plates.” Traum stated it best:

“A considerable number of ‘exact’ methods of analysis for folded plates are known. Although there are differences between various methods as to the amount of work required to arrive at the final magnitude of all stresses and deflections, still most of them require considerable time and effort. Even methods of particular loadings or slope deflections, which are far less laborious than solutions using simultaneous differential or linear equations, necessitate a repetition of the complete analysis of the folded plate structure for several particular cases of loadings. It is therefore not surprising that many authors advocate approximate solutions.”

In a 1964, E. Traum presented a simplified, yet “exact”, procedure for the analysis of prismatic folded plates using the method of moment distribution. In this method, the ridges are first considered as unyielding supports for the calculation of all transverse moments in the slab. Then they are subjected to unknown loads which constitute the true slab reactions, taking into account the settlement of the ridges. One single moment distribution is sufficient to express the relationship between those reactions. A set of linear simultaneous equations yields their “exact” values. A slight discrepancy, not exceeding 3 to 5 percent, exists between these solutions and the theoretically exact solutions. This is due to the use of the first term of a Fourier series to express the ridge loading on the structure. However, this discrepancy is negligible in view of loading assumptions and physical properties which are far less accurate. The main advantage of this procedure, compared with other methods is in the fact that by subjecting the ridges directly to unknown loads, all factors affecting their settlement are immediately considered.

Ultimate Strength Method of Analysis

The second method of analysis developed based on ultimate strength where prestressed concrete is seen to behave in a manner similar to mildly reinforced concrete. Thus a second family of formulas was derived taking the shape of $M = A_s f_{jd}$. Later, this approach was extended to include elastic behavior by expressing the internal resisting moment as a tension-compression couple even within the working range.
concrete distribution in ultimate strength design using the Whitney method for predicting ultimate moment capacity.\textsuperscript{17}

Load-Balancing Method for Prestressing

T.Y. Lin in 1961 introduced the concept of load-balancing for prestressed concrete structures.\textsuperscript{18} Later, in 1963, Lin wrote a detailed paper in which an application of load-balancing for folded plates was discussed.\textsuperscript{19} While load-balancing method often represents the simplest approach to prestressed design and analysis, its advantages over the two previous methods is not significant for statically determinate structures. When dealing with statically indeterminate systems, such as folded plates, the load-balancing method offered tremendous advantages both in calculations and in visualization.

An application of three-dimensional analysis for folded plates was typically required. Complete load-balancing in all directions could not easily be achieved for folded plates. Cables could be post-tensioned along the shell surface as that the vertical component would balance the gravity load. For a long shell behaving in accordance with the beam theory, a cable with a parabolic vertical projection would counteract the beam action. Practically no deflection along the length of the shell existed and the stress distribution was essentially uniform. Bending stresses would still exist in the transverse direction of the shell, but they could be analyzed as strips of arches supported at different points. For the unbalanced load (generally the live load or a portion of it), ordinary shell analysis could be applied to determine the stresses. Deflections were found to practically be negligible.\textsuperscript{20} This load-balancing method transforms a prestressed structure into a nonprestressed one subjected only to the unbalanced portion of the loading. This method presented a realistic approach which helps the engineer to visualize the effect of prestressing. And in T.Y. Lin’s own words, “opens up a new frontier in the analysis and design of prestressed concrete structures.”\textsuperscript{21}

RESEARCH AND TESTING

Even with the high demand and use of prestressed/post-tensioned folded plate, research and testing started over a decade after the first use of this system. In the early 1960’s, numerous research projects involving the testing of small scale and full scale folded plates occurred. The following paragraphs illustrate some of testing done during this time period. Most of the testing during this time indicated that the engineering assumptions made in analysis and design of folded plates up to this time period was quite accurate, typically within 5 percent of the theoretical values, or conservative.

Small Scale Testing

One example of research was performed by D. Billington and R. Mark in 1965. Rather than design models which serve to aid the designer’s judgment in dimensioning a particular structure, using a small scale model and a goal of understanding as completely as possible the behavior of the folded plates to derive general mathematical formulations. The research included both experimental (physical models) and theoretical (mathematical models) studies. The design of concrete thin shell structures has commonly been based on a mathematical analysis which assumes elastic material behavior. While it is highly
desirable to use an analysis based on the true inelastic characteristics of reinforced concrete, at that time such an analysis was not available for most thin shell systems.\textsuperscript{21}

Another example of research was carried out in the Department of Civil Engineering of the University of Toronto where emphasis was placed on the determination of the most economical tie spacing and the over-all performance of the unit. The plastic plate model analysis was succeeded by a behavioral study of a 7 foot (2.1 m) wide and approximately 48-foot (14.6 m) long unit which was precast and post-tensioned. The performance of the post-tensioned unit was in close agreement with the behavior predicted by the plastic model; in particular the tie forced as established by the model compared favorably with the tie forces measure on the prototype.\textsuperscript{22} Three possible modes of failure were anticipated in the structure of this type: (1) cracking on account of diagonal tension near the supports; (2) failure of the unit due to bending in the longitudinal direction (failure of the concrete in compression or overstressing of the wires); and (3) failure due to transverse bending of the unit.

The model showed that the unit displayed “beam action” in the longitudinal direction making possible the determination of the (longitudinal) bending stresses. Hence, major interest was given to the load carrying capacity of the unit in the transverse direction and, in particular, the question of adequate safety of the unit against fracture due to diagonal tension. The interesting part of this research was that the unit did not show hairline cracks near the supports. In fact, no cracks were visible at the intersection of the stem and the inclined slabs.\textsuperscript{23}

\textbf{Full Scale Testing of Pretensioned Folded Plate with Lightweight Concrete}

Three full-scale pretensioned folded plate roof units were tested to collapse by J.I. Glanville in 1963.\textsuperscript{23} To more fully utilize the potential value of the folded plate as a roof unit, a 2-1/2 inch (6 cm) thick light-weight aggregate concrete slab was used and pretensioned with draped prestressing stands; engineers had already been using this type of system in buildings prior to this time. The units were precast in a V-shaped mold, thereby providing a trough less susceptible to leakage than a similar structure cast in the inverted position and required field-jointing at the trough. The field joints for the present units would be made at the peaks in an actual roof structure. Stiffening diaphragms were not provided at the supports as they were not required for a folded plate of this simple geometric shape. Designs based on early by a folded-plate theory yielded results similar to those of the simple equivalent beam theory and consequently the latter method was used in the analysis of the test units. In diagonal tension calculations, it was assumed that the vertical shearing force at a section is resolved into two components parallel to the sloping side of the unit. The theoretical ultimate moment was calculated on the basis of the rectangular stress-block theory for the equivalent beam, and on the basis of $f'_{c}$ values estimated from the age of the concrete and curing conditions.\textsuperscript{24}

Tests results for a folded plate of this simple geometric form may be predicted with reasonable accuracy by assuming an equivalent beam; this assumption being equally applicable to units in light-weight and normal weight concrete with straight or draped strands. The research indicated that the absence of a stiffening diaphragm does not impair the strength of the unit. It is also interesting to note that the units tested without lateral tie bars reached the predicted ultimate moment without any appreciable secondary buckling of the free compression edges taking place; and therefore, for units within the slenderness
range of the test units, no special provisions need be made for this buckling effect. Tests serve to justify the use of the simple beam theory for the flexural analysis of the simple folded plate shape investigated. Further tests making use of strain reading to obtain the prestress loss and to provide information about principal stresses in regions of complex stress distribution, such as at draping points, supports and points of load concentration, could be used to an advantage in further analyzing the behavior of the folded plate unit.  

BUILDING CODES, STANDARDS & INSTITUTIONS

Many developments in the prestressing industry, which include post-tensioning, industry, were occurring during the late-1950’s to early-1970’s in the United States. The Prestressed Concrete Institute (PCI) was chartered in Tampa, Florida in 1954. In 1958, the first specification on pretensioned concrete published by Harry Edwards. In addition, ACI-ASCE Joint Committee 323 (later 423) published Tentative Recommendations for Prestressed Concrete. The ACI 318-63 Building Code recognized prestressed concrete in 1963 by including it in the building code. ACI Committee 347 Recommended Practice for Concrete Formwork included a section on folded plates, thin shells, and long span roof structures. In 1971, the First Edition of PCI Design Handbook published. Unfortunately, the Post-Tensioning Institute was not established until 1976 after the folded plate movement in the United States had stopped.

CONCLUSION

From the late 1950’s to the early 1970’s, architectural magazines were full of examples of folded plate structures built by many different designers. Then the construction of folded plates seemed to stop in the United States. Possible reasons why folded plate construction stopped in the United States are numerous, but the two main factors are the architectural solution, and the lack of understanding of folded plate structures by engineers and architects. At the present, with computer analysis more precise, and construction estimating less laborious, the design of folded plates are less time consuming. In the words of Milo Ketchum, “Some time in the future, the cost of structural steel will rise beyond reason, and some one will discover the utility and beauty of shell structures, will design them, find they are salable, publicize them, and will start the cycle of popularity again. Things are not built or done because they are economical, beautiful, or utilitarian. They are built or done because someone wants to build or do them, and in the process then become economical or beautiful or utilitarian.”

2 Winter, G., Pei, M., 1947 “Hipped Plate Construction,” *Journal of the American Concrete Institute*, V. 18, No. 5, pp. 505 – 531.
3 www.ketchum.org/shellpix.html.
5 Peterson, J., 1954 “History and Development of Precast Concrete in the United States,” *Journal of the American Concrete Institute*, V. 25, No. 6, pp. 477 – 496.
Figure 1: Schematic of a folded plate. Image courtesy of the American Concrete Institute.

Figure 2: Typical Russian Coalbunker – Folded Plate Floor System. Image courtesy of the American Concrete Institute.

Figure 3: A folded plate for a gymnasium in Colorado by structural engineer Milo Ketchum. The folded plates are the typical two-element construction. North light is achieved in this roof structure by truss elements cast in the plates. The edge plates are small because they are supported by columns. Photograph courtesy of Mark Ketchum.
Figure 4: Langendorf Bakery in Los Angeles, CA. A portion of the roof with reinforcing in place. Freyssinet anchorage cones placed in holes through the outside walls. Temporary bulkheads in front of cones form dam for anchorage block. Note the prestressing cables are straight. Photograph courtesy of the American Concrete Institute.

Figure 5: Langendorf Bakery in Los Angeles, CA. Prestressing operation: two jacks were used for each cable, stressing simultaneously from both ends. Man on scaffold measures elongation of cable wires. Photograph courtesy of the American Concrete Institute.
Figure 6: Mackey Airline Hangar at the Broward County International Airport in Fort Lauderdale, FL. Photograph courtesy of Portland Cement Association.

Figure 7: Washington State University circular dining hall. Photograph courtesy of Washington State University Libraries, Manuscripts, Archives & Special Collections.

Figure 8: Longitudinal radial joints. Image courtesy of Precast/Prestressed Concrete Institute.
Figure 9: Circumferential joint connections. Image courtesy of Precast/Prestressed Concrete Institute.

Figure 10: Cloverleaf Lanes Bowling Alley, Dade County, FL. Photograph courtesy of Precast/Prestressed Concrete Institute.

Figure 11: Cloverleaf Lanes Bowling Alley, Dade County, FL: Rear wall of bowling lanes showing portable jacking unit in use on roof. Photograph courtesy of Precast/Prestressed Concrete Institute.
Figure 12: Cloverleaf Lanes Bowling Alley, Dade County, FL. Placing concrete – double forms. Photograph courtesy of Precast/Prestressed Concrete Institute.

Figure 13: Cloverleaf Lanes Bowling Alley, Dade County, Florida. End Anchors for cables. Photograph courtesy of Precast/Prestressed Concrete Institute.

Figure 14: Cloverleaf Lanes Bowling Alley, Dade County, Florida. Prestressing cables. Photograph courtesy of Precast/Prestressed Concrete Institute.
Figure 15: Allegheny Airline Hangar, Boston, Massachusetts. Post-Tensioned folded plate solution. Photograph courtesy of Precast/Prestressed Concrete Institute.

Figure 16: Allegheny Airline Hangar, Boston, Massachusetts. View of the complete folded plate roof. Photograph courtesy of Precast/Prestressed Concrete Institute.
Figure 17: Allegheny Airline Hangar, Boston, Massachusetts. Jacking End Anchorage and Non-Jacking End Anchorage. Photograph courtesy of Precast/Prestressed Concrete Institute.

Figure 18: Allegheny Airline Hangar, Boston, Massachusetts. Tendon placement in folded plates. Photograph courtesy of Precast/Prestressed Concrete Institute.
Figure 19: Allegheny Airline Hangar, Boston, Massachusetts. Concrete placement on an inclined folded plate. Photograph courtesy of Precast/Prestressed Concrete Institute.

Figures 20: Test specimen during application of first loading. Unit after failure. Photograph courtesy of American Concrete Institute.
Figures 21: Test specimen during application of first loading. Unit after failure. Photograph courtesy of American Concrete Institute.
The Development of Unbonded Post-Tensioning Tendons used in Parking Structures in Deicing Salt Regions

by C. Walker

Synopsis: This paper relates the historical development and use of single strand unbonded post-tensioning tendons in parking structures constructed in deicing salt regions in North America since the early 1960’s. These parking structures were constructed in southeast Canada and the northeastern and midwestern United States using wire and strand tendons as the primary flexural reinforcing and were exposed to deicing salts during the winter. Parking structures exposed to ocean salts have not been reviewed. Most of the following discussion, however, will also apply to them.
CARL WALKER grew up in Duluth, Minnesota and is a graduate of the College of Engineering of The University of Michigan. In 1965, he founded Carl Walker & Associates, Inc., now Walker Parking Consultants, Inc., Consulting Engineers and Parking Consultants, in Kalamazoo, Michigan. This firm is now the largest consulting firm in the world specializing exclusively in the planning, design, and restoration of parking structures.


Also, in 1996, Mr. Walker was a co-founder of the Carl Walker Construction Group, Inc., Pittsburgh, Pennsylvania, which specializes in the construction and restoration of parking structures in the northeast United States.

In 2000, Mr. Walker formed CWConsultants, LLC and is currently active as an independent parking consultant and structural engineer throughout North America.

He is an active member of ACI Committee 362—Parking Structures and ACI Committee 423—Prestressed Concrete. He is also active on the Parking Structures Committee of the Precast Prestressed Concrete Institute, the Parking Consultants Council of the National Parking Association, and the International Parking Institute.

INTRODUCTION

The early 1960’s saw the acceptance of long span prestressed concrete in parking facilities. Long span prestressed concrete allowed the structure to economically clear span parking modules of 50 to 65 feet and beyond if necessary. This eliminated the fender-bender stigma of having to drive around columns between the parking spaces, and also provided the flexibility of future restriping of parking spaces as passenger vehicle sizes changed over the years.

While built on building sites and with the design governed by building codes, the majority of modern parking structures are designed to be open on the sides or facades to provide natural ventilation, and thus exposed to climatic temperature changes, wetting and drying from rain and snow melt, and freeze/thaw action during the winter months in the North. Parking structures in the North are also exposed to deicing salts carried into the facilities by passenger vehicles. In reality, the parking structure has similar exposure conditions as a highway bridge without the heavy truck loadings. Thus a parking structure is not truly a building and not truly a bridge—it is unique in that it must be designed to accommodate the exposure conditions of a bridge while being governed by building and zoning codes.
The concepts of prestressed concrete were developed in Europe in the first half of the 20th century and were brought to this country after World War II in the late 1940’s by Professor Gustav Magnel of the University of Gent, Belgium. Many experiments had been made since 1886 in the North America with R. E. Dill of Alexandria, NB applying for a patent in 1925 for the manufacturing of prestressed precast concrete posts and slabs by post-tensioning the steel. The first major prestressed concrete structure in the United States was the Walnut Lane Bridge which was constructed in Philadelphia in 1949-50 using precast post-tensioned beams with cast-in-situ slabs. About the same era, the Oneida Lake Bridge near Syracuse, New York was constructed using post-tensioned prestressed concrete beams.

Precast prestressed concrete was introduced in the same era with the Prestressed Concrete Institute being formed in 1954. This group represented both the precast prestressed concrete manufacturers and the post-tensioning tendon manufacturers.

Before World War II, parking structures were designed structurally similar to warehouse buildings and were typically operated as attendant parking facilities. Vertical access in these buildings was accomplished with sloped vehicular speed ramps or elevators. However, a few fully sloping floor parking structures existed at this time such as the Fort Shelby Garage in Detroit that was designed by Albert Kahn and Associates in 1926. In 1950, the Battery Parking Garage in New York City was a long span structure with bays of 32 feet by 58 feet and used air-entrained concrete. Also, in the 1950’s, a number of clear span mild steel reinforced concrete parking structures were designed by the Detroit firm, National Planning, Inc., and used pan joists or one-way flat slabs spanning about 20 feet between haunched beams which clear spanned the parking bays up to 55 feet. During this era, Prestressed Concrete of Colorado built the first roof parking deck using precast prestressed single tees for the Spitzer Electric Company. The clear span varied from 17 feet to 80 feet.

The first long span prestressed post-tensioned concrete parking garage was designed in the early 1960’s by T. Y. Lin & Associates and constructed in Beverly Hills, California. This used the spread single tee system where six feet wide by three feet deep precast pretensioned single tees spanning approximately 64 feet were erected on precast columns spaced at approximately 25 feet on center. A post-tensioned composite concrete flat slab spanned between the single tees. In addition, post-tensioning tendons were cast into the precast single tees. The one inch gaps between the ends of the single tees and the column were grouted. The tendons in the tees were coupled through the columns with large (about three inch diameter) threaded studs. After the slabs were cast and post-tensioned, the tendon-stud assemblies in the single tees were post-tensioned to create a rigid frame. In the 1960’s, a number of clear span parking structures were built using the spread single tee rigid frame concept; mainly in the West and Midwest. See Photo 1.

Cast-in-place concrete construction contractors found that part of their market was being taken by the expanding use of precast concrete and they rapidly developed new long-span
forming and concrete placing systems to become competitive with the long span precast concrete systems. Expandable steel joists slab form supports and “flying forms” were developed. Concrete conveyors replaced the Georgia buggy and skip hoists. Later, concrete pumps were developed. These, along with post-tensioning, made the construction of cast-in-place long span structures more competitive with the long span precast concrete systems.

At the same time, in the late 1960’s, precast manufacturers were converting to the 8 foot wide by 24 inch deep double tee as their primary long span member for construction of parking structures. These later increased in size so now the 12 foot wide by 32 inch deep double tee is commonly used with precast double tees up to 15 feet wide available in some areas of North America.

Thus by the early 1970’s, the clear span parking structure was typically being built with either a long span cast-in-place post-tensioned concrete frame or with double tees erected side-by-side with a 3 inch composite concrete topping slab. Later, pretopped or traffic-direct double tees with 4 to 5 inches flanges were often used where shipping was not a major cost factor.

In summary, prestressed concrete, either precast pretensioned or cast-in-place post-tensioned, took over the new parking structure market because of its efficient use of concrete and the clear span benefits on the interior of the structure.

DEICING SALT USE ON ROADWAYS

In the 1950’s, in the winter climate regions of the United States, the Bureau of Public Roads, now the Federal Highway Administration, instituted a “bare roads policy”. This policy stated that roads that were constructed using federal funds had to be maintained in the winter in a bare condition—driving on packed snow or sanded packed snow was not allowed. Rock salt (sodium chloride) was the most economical way to achieve a bare road condition. It is inexpensive and abundantly available and has become the primary deicer used on roadways, outdoor pedestrian routes and other areas where deicing is desirable for traction and safety. Liquid salt brine has also been used successfully in some areas. From the mid 1950’s to the mid 1960’s, the use of salt as a deicer on the roads in the United States increased from about 2 million tons per year to over 12 million tons per year. 15 millions tons per year are now used in the United States according to the Salt Institute. Canada uses 4-1/2 to 5 million tons per year.

Salt is highly corrosive to ferrous metals (steel) and in the 1960’s, corrosion of cars and trucks became very evident, annoying, and economically expensive. Concrete spalling (pot holing) and delamination of highway bridges and pavements was the first evidence of the deteriorating effects of the use of deicing salts. See Photo 2. By the 1970’s, highway bridge deck replacement projects were common. Later, parking structures also began to deteriorate due to salt carried in from the roadways by parking vehicles. Some of this deterioration was so severe that parking
structures less than 10 years old were requiring major repairs and restoration, and sometimes demolition.

DEICING SALT AND CONCRETE STRUCTURES

It has long been known that salt in the form of sodium chloride and calcium chloride is highly corrosive. When the bare roads policy was put into effect, crushed rock salt (sodium chloride) was the most economical material available to achieve the required removal of snow and ice. Not much thought was given to the possible deteriorating effects of deicing salts on concrete structures and particularly the slabs. The conventional wisdom was that concrete was impervious and thus the deicing salt water would simply run off into the storm drainage system. Soon, reinforced concrete bridge decks began spalling and delaminating due to deicing salt induced corrosion of the reinforcing steel. Because of the deicing salts carried into parking structures by the parking vehicles, parking structure floor slabs soon began to deteriorate, particularly those constructed of reinforced nonprestressed concrete.

Concrete is normally protective of the ferrous reinforcing steel. Bare steel will corrode or rust in the presence of water and oxygen. However, when steel is in concrete, the high alkali concrete forms a passive protective layer around the reinforcing steel and electrochemically impedes the corrosive action of water and oxygen. The cause of the deterioration of the bridge decks and parking structure slabs was the migration or diffusion of the chloride ions through the concrete to the reinforcing steel. The chloride ions broke down the passive protective layer around the steel by lowering the pH in the concrete, and corrosion commenced. When steel corrodes, it expands. The expansive pressures create internal bursting forces in the concrete. This causes the concrete to fail in tension and creates surface spalls and internal delaminations. Once a spall or delamination is initiated, the void quickly fills with deicing salt water and deterioration accelerates.

To prevent this corrosion deterioration phenomenon, the first reaction was to seal the bridge decks with a 50/50 mixture of boiled linseed oil and mineral spirits (kerosene, gasoline, or diesel fuel). Because linseed oil is a paint derivative, repeated applications caused the decks to become slick and loose skid resistance. In parking structures, the linseed oil application gave way to penetrating sealers—epoxy and polyurethane.

Nonprestressed reinforced concrete parking structures were the first to deteriorate. Those with thin slabs—pan joists and waffles slabs—deteriorated quickly. Tensile cracking of the concrete is a fact of life of reinforced concrete due to concrete shrinkage and flexural bending. Shrinkage cracks and tensile cracks in the negative moment regions on top of the slabs provided direct paths for the deicing salt water to migrate to the reinforcing steel and initiate corrosion with subsequent delaminations, spalling, and steel failure.

Cast-in-place post-tensioned concrete slabs did not have the endemic cracking due to shrinkage and flexure and thus seemed to be much more durable. However, in due time, tendons began failing as indicated by strand eruptions from the end anchorages or out of
the slab top or bottom surface of the slabs. It became apparent that water and deicing salt water was migrating to the prestressing steel and causing corrosion.

It should be noted that the anchorage assemblies of unbonded tendons are the only elements of the tendons that are in direct contact with the concrete. Thus the natural protection of the steel tendons by the concrete is not available and protection of the tendon wires or strand can only be achieved by the sheathing and the grease.

The early tendon sheathings were Sisal Kraft paper. Sisal Kraft paper is fiber reinforced and coated with plastic on one side and used in rolls about 2-1/2 inches to 3 inches wide. The paper wrap was to prevent bonding of the concrete to the steel and was not waterproof. The primary purposes of the grease were to lubricate the steel to reduce friction during tensioning of the tendon and act as a corrosion inhibitor. Early greases were mainly of automotive origin and were later found to be susceptible to oxidation and emulsification, and would loose their corrosion resistive qualities.

Finally, it became apparent that the design details of the structure and quality of constructing the details had much to do with the deterioration of parking structures. Shrinkage cracks over tendons which had insufficient cover allowed deicing salt water to quickly migrate to the tendons. Failure to seal construction joints and failure to rout and seal concrete cracks that occurred during or following construction allowed deicing salt water to migrate to the tendons or anchorages. Drainage details which allowed the deicing salt water to flow over the anchorage zone edge of a slab also led to failures.

Thus, in the 1970's, it became apparent that the crack-free nature post-tensioned concrete was not the durability panacea that it was thought to be.

**POST-TENSIONING TENDON DEVELOPMENT**

In the United States, early unbonded post-tensioning systems used high strength stress relieved steel wires—BBRV, bars—Stressteel, or strand—Atlas.

1. **BBRV**—A Swiss system that was developed by three structural engineers, Birkenmeier, Brandestini, and Ross, and Vogt who was a mechanical engineer. Vogt developed the machinery that cold-upset the ends of the wires to create the button-head anchors. The system was licensed to North American tendon manufacturers. It used ¼ inch wires bundled together, greased, and wrapped with Sisal Kraft paper as sheathing. The wires ran through a stressing head and were “button headed” to anchor them. The post-tensioning elongation was held with shims or a threaded nut. Emphasis was placed on the positive anchorage of the button heads.

2. **Stressteel**—A solid bar system with diameters of 7/8 inches to 1-3/8 inches, greased and wrapped with a Sisal Kraft paper anchored with a threaded nut or wedge anchors.
3. **Atlas**—This system was developed by Edward K. Rice and others at T. Y. Lin & Associates and used 1/2 inch seven-wire prestressing strand which ran through a anchorage assembly manufactured of coiled wire and a plate and anchored with two half wedge chucks. The strand was also greased and Kraft paper wrapped. This system was commonly used on the West Coast of the United States until the 1964 Anchorage, Alaska earthquake when there were a number of failures associated with the system.

After the Anchorage, Alaska earthquake, Atlas redesigned their anchors to be ductile iron castings because the original anchor was banned by the City of Los Angeles. At the same time, the post-tensioning manufacturing industry scurried to find an anchorage system to match the economies of the Atlas system, yet provide assurance of maintaining the integrity of the anchorage of the strand during earthquakes. The eventual evolution was the development of single strand ductile anchor castings. At the same time, the ACI 318 building code was changed to require back-up flexural mild steel reinforcing in addition to the primary post-tensioning tendons. The Uniform Building Code, which was used in the western seismically active United States, required the mild steel reinforcing to carry the flexural dead load plus 25% of the live load on an ultimate strength (unfactored) basis.

One of the problems with the wire-button head system was that the anchorage bearing plate was placed against a pocket form at the slab edge, and the stressed anchor head was covered with a subsequent concrete cap—thus creating a cold joint between the slab concrete and the cap concrete. Often, this cold joint was not sealed and deicing salt water would migrate through the joint to the prestressing wires and initiate corrosion. Sometimes, the wires between the anchorage plate and the stressing head would be completely rusted through but the wires would be “rust anchored” in the anchorage plate and the tendon would not lose its tensioning force. This, of course, is not structurally acceptable and has required a number of expensive repair projects in the late 20th Century.

By the mid 1970’s, the single 7-wire strand systems with ductile iron casting and wedge chuck anchorages had taken over the unbonded post-tensioning market. However, the types of grease, and the method of greasing and sheathing remained quite variable.

**Greases**—Greases used in the manufacture of tendons had two purposes—first, the grease was to lubricate the tendon to reduce friction during the tensioning process; second, the grease was to act as protection against corrosion. It was found that automotive lithium based grease served these requirements well and was readily available. However, these greases would emulsify in the presence of water and oxidize in air. This would happen over time and thus, did not inhibit the tensioning process, but the emulsification and oxidation did reduce the corrosion inhibiting properties of the grease. Also, during tensioning, grease would rub off sections of the strand leaving it exposed for possible corrosion.
Walker

Prescon Corporation, one of the early BBRV licensees, used a product known as Prescon Lube, a heavy tar-like grease with asbestos fibers, which performed better than some of the automotive greases.

Incorporation of corrosion inhibiting chemicals into the grease started in the mid to end 1980’s concurrently with the development of encapsulated tendons and are being used effectively today.

**Sheathing**—Early sheathing of the strand was crude at best. The tendons were greased by hand, and then wrapped by hand in a spiral fashion with Sisal Kraft paper so the tendon would not bond to the concrete. This was a tedious process. It was not waterproof and did not provide any long term protection for the steel tendon.

One sheathing system that later proved to be quite susceptible to corrosion and failure was the push-through tendon. Semi-rigid plastic tubing was laid in a long trough. The strand from a reel was run through a greasing device and then pushed through the tubing. In theory, the grease was to fill the annular space between the strand and the wall of the plastic tube, but in actuality, the grease never did completely filled the void.

Often, the plastic tube sheathing on the tendon would get ripped or damaged during shipping, storage, or placing and rain water or excess concrete water would enter the ungreased space between the tube wall and the strand. Even with taping prior to concreting, it was impossible to eliminate the possibility of water entering the tendon at some point in the construction process. Unfortunately, many future failures occurred with tendons manufactured with this sheathing system.

Atlas and others later developed a “cigarette wrapped” plastic sheathing. A machine was developed which took strand from a reel, ran it through a greasing apparatus, and rolled a 40 mil thick ribbon of plastic sheet around the strand; then the edges of the rolled plastic ribbon were heat sealed with a flame to form the completed sheathing. This greatly reduced the labor cost of producing single strand tendons. However, this equipment was very operator sensitive. The heat sealed lap joint of the sheathing often failed; particularly when placing the tendons in cold weather. If observed before concreting, these failed lap sheathing joints could be taped; however, sheathing failures were not always seen during inspections.

In the 1980’s, a 40 mil plastic sheathing extruded monolithically around the greased strand was developed. This solved the lapped joint problem in the sheathing; however the sheathing still was quite susceptible to damage during storage, shipping, placing and concreting which allowed direct exposure of the strand to the elements and future deterioration. However, thickness uniformity of the sheathing was difficult to maintain. In the mid 1990’s, parking structure design engineers began specifying a 50 mil extruded sheathing which reduced the sheathing thickness tolerance and damage problems.

**Center-Stressing**—Stressing of the anchorages at the edges of slabs often entailed working in rather precarious positions with heavy equipment high above the ground. To
alleviate this, “center-stressing” anchorage systems were developed which allowed the stressing of the tendons to be done away from the edges of the slabs in much more efficient working conditions. Center-stressing required a “block-out” of the slab to provide access of the jacking equipment to the tendons. The block-out was filled with concrete after the jacking was completed.

The problem with center-stressing was the concrete fill of the stressing block out created a construction or cold joint between the slab concrete and the fill concrete. If this joint was not sealed, or if the seal failed, deicing salt water easily migrated through the joint to the tendon and initiated corrosion and subsequent tendon failure.

ENCAPSULATED TENDONS

As unbonded tendons were increasingly used in parking structures built in deicing salt regions, it became apparent that more protection was needed in addition to the corrosion resisting grease and the continuous sheathing over the strand. Failures were still occurring due to corrosion in the vicinity of the anchorages. Typically, the sheathing didn’t cover a short one to three inch section of strand inboard of the dead end anchorages. Deicing salt water would enter the stressing pockets at the stressing ends due to no or poor pocket grouting, or poor slab edge storm water run-off details.

Plastic “trumpets” or connectors were developed to cover the interface between the end of the plastic sheathing on the strand and the inside face of the anchorage casting. Initially, the trumpet friction fit tight to the anchorage, and the other end of the trumpet was sealed to the sheathing with plastic tape. Often, the anchorage-trumpet connection became detached due to normal construction abuse. Consequently, the trumpet was revised to be a positive threaded connection to the back of the anchorage, and a neoprene grommet was inserted on the other end of the trumpet to provide a seal between the trumpet and the strand sheathing. Later, the trumpets were specified to be of clear or translucent plastic so the adequacy of grease filling the annular space between the interior of the trumpet and the strand could readily be observed. If the grease in the trumpet was inadequate, additional grease was pumped in with a needle similar to the air needle for a football or basketball. See Photos 3, 4 and 5.

Corrosion of steel is an electrochemical reaction requiring in addition to the steel, oxygen and water. Since unbonded tendons are not in direct contact with the concrete except in the vicinity of the anchorages, the passive or protective layer adjacent to reinforcing steel cast in concrete is not created. Thus, there is no protection of the steel post-tensioning strand other than the integrity of the sheathing and the grease coating—both of which can be easily compromised. However, the anchorages are in direct contact with the surrounding concrete and provided an opportunity for the tendon to establish an “electrical circuit”. Thus, the electrically isolated tendon was developed. In addition to isolating the strand from the concrete, as previously discussed, the anchorage casting is coated with epoxy or plastic, and the ends of the cut-off tensioned strand are capped to completely isolate or insulate the tendon from the surrounding concrete. This system is
patented. However, in the late 1980’s, generic plastic coated anchorage castings with end cover caps became available and became widely used.

Following the Anchorage, Alaska earthquake in 1964 and the associated post-tensioning anchorage failures, the Building Code Requirements for Structural Concrete (ACI 318) were revised to require a minimum amount of bonded reinforcement. ACI 318-05 states in the Commentary, “Some bonded reinforcement is required by the code in members prestressed with unbonded tendons to ensure flexural performance at ultimate member strength, rather than as a tied arch, and to limit crack width and spacing at service load when concrete tensile stresses exceed the modulus of rupture.” In addition, the Code states in the Commentary, “Research has shown that unbonded post-tensioned members do not inherently provide large capacity for energy dissipation under severe earthquake loadings because the member response is primarily elastic.”

The Code Section 18.9.2.2 states, “Bonded reinforcement required …..shall be uniformly distributed over precompressed tensile zone as close a practicable to the extreme tension fiber.” This means that the required minimum bonded reinforcement must be placed in the same locations as if the slab was not prestressed—in the top of the slab over the supports and at the bottom of the slab at midspan. Thus, the mild steel in unbonded post-tensioned slabs is exposed to the same climatic elements including deicing salts as typical non-prestressed slabs, with the exception that the post-tensioned slabs generally do not have as many, if any, flexural cracks which provide avenues for the salt water and chloride ions to migrate to the level of the reinforcing steel.

In order to reduce the potential of corrosion in prestressed concrete, ACI 318-05, Section 4.4, requires that the maximum water soluble chloride ion in the concrete be 0.06 percent by weight of the cement. In non-prestressed reinforced concrete, this limit is increased to 0.15 percent.

ACI 318-05, Section 7.7.2, provides the cover requirements for prestressed cast-in-place concrete exposed to the weather (parking structures should always be considered exposed to the weather) of one inch. Section 7.7.5.1 further states that for prestressed members exposed to corrosive environments and with certain flexural stress conditions, the minimum covered should be increased 50 percent, or in the case of slabs increased to 1-1/2 inches. However, the Commentary R7.7.5 recommends a cover of 2 inches where concrete is exposed to deicing salts in service. This would apply to all floor slabs in parking structures in deicing salt regions.

CERTIFICATION

Due to some quality problems with manufactured tendons in the early 1980’s, the Post-Tensioning Institute developed the, “Specification for Unbonded Single Strand Tendons.” In the late 1980’s, to enforce the Specification, PTI developed a certification program for unbonded tendon plants modeled somewhat after the Prestressed Concrete Institute’s plant certification program. “PTI Certified Plants” were required to have stringent quality control procedures and were inspected at random times at least twice a year by an
independent agency to help assure compliance of the post-tensioning systems with required quality control procedures and the Specification. The Specification later became, by reference, a part of the ACI 318 building code requirements. The specification covers materials, performance, certification and installation.

UNBONDED VS. BONDED POST-TENSIONING SYSTEMS

The major advantage of bonded post-tensioning is the tendon is bonded to the structure by grout inside the tendon sheathing and thus performs similar to a mild steel reinforced concrete beam for ultimate strength and less mild steel reinforcing is required. The primary disadvantage is the grouting process is an added step and is problematic. Incomplete grouting, ungrouted tendons, grout segregation, and frozen grout are examples of bonded tendon problems. Also, the grouting is weather sensitive requiring heating of the construction area if grouting is to be done during the winter.

The major disadvantage of unbonded tendons is the requirement for additional mild steel reinforcing to supplement the ultimate strength, particularly for seismic events. The is often offset by the advantages which include the ease of inspection, the lack of weather sensitivity, and once the concrete is placed and the tendons stressed, the job is complete—no grouting has to be done.

OTHER RECOMMENDATIONS TO IMPROVE DURABILITY

Other factors, in addition to encapsulating the tendons and providing adequate cover, which contribute to the durability of post-tensioned slabs for parking structures in deicing salt regions include:

1. Positive drainage—Two percent slope (1/4 inch per foot) is desirable; one percent minimum slope (1/8 inch per foot) as constructed.

2. Low porosity concrete—Water/cementitious ratio less than 0.40; using slag aggregates; using fly ash and/or silica fume as a Portland cement replacement. Silica fume concrete, while quite impervious, has a history of being difficult to properly finish. Often, using other “pore fillers” such as slag aggregates and fly ash along with a low w/c ratio is adequate.

3. Epoxy coated bonded reinforcement or other corrosion resistive bonded reinforcement.

4. Air entrainment per ACI 318.

5. Corrosion inhibitors—Calcium nitrite is the most common. Pore blocking inhibitors do not work well in micro cracked concrete.

7. Good joint detailing—Use of silicone sealants on the roof have proven to be ultraviolet (sun) light resistant.

8. Good placing, finishing and curing practices—Under-finishing and over-curing is the rule—minimize the finishing of the concrete and maximize the curing of the concrete. Moist curing for at least 7 days is desirable when practical. Application of a membrane curing compound after the moist cure is a good practice.

9. Preventive maintenance—A disciplined preventive maintenance program will add considerably to the service life of a parking structure.

Often, engineers will reduce the maximum cover if epoxy coated bonded reinforcing steel is used. This has proven to be a false economy, particularly since the minimum amount of bonded reinforcement required by ACI 318-05, Section 18.9.2 is not determined by the “d” distance but by the “area of that part of cross section between the flexural tension face and center of gravity of gross section.” Also, this section states, “Bonded reinforcement required …..shall be uniformly distributed over the precompressed tensile zone as close a practicable to the extreme tension fiber.” Thus, “close as practicable” can be interpreted as a minimum of 2 inches of cover in deicing salt regions.

THE SERVICE LIFE APPROACH TO DURABILITY

Service life of a parking structure can be defined as that period of time that a parking structure can be expected to last without requiring major repairs. Major repairs are defined as those repairs requiring the use of a jack-hammer for concrete removal and the of replacement concrete and/or reinforcing. This is assuming that periodic (at least annual) preventive maintenance is carried out on the structure. An expected 60-year service life is not unreasonable with today’s parking structural design, materials, and construction technologies.

Today, estimating service life of parking structures is at the best an “educated guess.” However, computer programs have been developed and are being refined using research in diffusion of chemicals into concrete and other advanced corrosion technologies which are making estimating service life much more accurate. Recent case studies have shown large cost savings in new construction using the service life approach to designing concrete mixes.

These computer programs are the most accurate for mild steel reinforced structures. Since the encapsulated post-tensioning tendon has different corrosion “hot spots” than mild steel reinforcing, namely the anchorages and sheathing damage areas, the service life is quite dependent on design details and construction practices. Thus the “belt and suspenders” approach to design and to estimating service life is still prudent. Parking structures designed and built according to the recommendations of ACI Committee 362-Parking Structures since the early 1990’s have shown marked improvement in durability and potential service life.
The primary source of deterioration of post-tensioned parking structures in deicing salt regions is the interaction of the deicing salts with the ferrous metals. One approach would be to cease using chloride based deicers. However, rock salt is currently the most economical deicing material. Thus, what can be done to better resist the corrosive effects of deicing salts in parking structures?

In Canada, heavy duty traffic bearing waterproof membranes are required to be installed over cast-in-place parking structure slabs, whether reinforced or post-tensioned. However, in time, the membrane wears off due to tire abrasion and the concrete slab surface is exposed to the weather and deicing salts if the membrane is not maintained.

Properly installed encapsulated post-tensioning systems are well protected against the migration of deicing salts. The main problem of deterioration of parking structure slabs is spalling and delamination caused by corrosion of the mild steel reinforcing which is required to be in the slabs by Code and by the failure to maintain the sealants at construction joints. Epoxy coated steel has had varied success in reducing deterioration of parking structure slabs. Its effectiveness is highly dependent on the placement of the steel—maintaining cover—and handling of the steel to minimize the “holidays” or chips in the coating which expose the raw metal. Uncoated stainless steel reinforcing has been tested in bridge decks. Composite bars using a stainless steel wrap over standard steel has been tried. Less expensive hybrid steels (MMFX) are being used. Also, stainless steel fibers to replace the reinforcing bars are being tested but must be approved by the Code writers.

Carbon fiber prestressing strand and other reinforcing are being tested in the laboratories. Carbon fibers would be unaffected by deicing salts. However, the main drawback is the low fire resistance of the carbon fiber tendons. Maybe, sometime in the future, a high temperature epoxy matrix which is used to bond the carbon fibers will be developed.

Galvanized reinforcing bars are available but their use is not wide spread.

Other coatings such as metallic ceramic coatings are being tested for use on both prestressing strand and bonded reinforcing steel. They protect the steel galvanically. Because these ceramic coatings are both rough and semi-porous, they will also bond well to the concrete.

SUMMARY & CONCLUSIONS

The first unbonded post-tensioned concrete parking structure slabs were constructed in deicing salt regions in North America in the early 1960’s using high strength button-headed wire tendons, greased, and wrapped with Sisal Kraft paper. These structures had very little bonded reinforcement. Primarily due to failures caused by earthquakes and deicing salt corrosion, improvements were made to engineering design methods, construction details, construction procedures, construction materials including the
unbonded post-tensioning tendons, and maintenance procedures which have made the encapsulated unbonded post-tensioned slab the cast-in-place parking structure slab of choice in the United States. The increased extruded sheathing thickness has created a rugged tendon for handling, shipping, placing, and concreting. Once the tendons are stressed and the anchorage caps put in place, the design is complete and sealed against the intrusion of deicing salts. In the winter construction season, the builder does not have to wait until warmer weather to perform grouting as there is no grouting to be done. While more bonded mild steel reinforcement is required with unbonded post-tensioning designs, this additional cost is more than off-set by the elimination of the need for the grout sheathing, the possible cold weather delays required for grouting, and the actual cost of grouting of the bonded tendons.

This discussion is primarily related to the durability of post-tensioned parking structures relating to their reinforcing systems. Parking structures are unique. Because prestressed concrete parking structures experience movement due to elastic shortening, creep, and shrinkage in addition to temperature changes, a key element of structural design is providing for the relief of forces and stresses caused by these movements. For a parking structure to achieve its intended service life, it must be designed and built for environment deteriorating effects—deicing salts—as well as restraint or relief of physical movement effects.

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Shannon is an architect living in Atlanta. A Master’s Degree in Architecture graduate from Yale, she has held academic positions at several universities including at the University of Nebraska. She has been employed by a number of architectural firms including the firm of Ross, Barney & Jankowski on the award winning Chicago school, The Little Village Academy. She is now in the final editing process of a book about the historical development of parking structures for the Urban Land Institute.

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Beverly Hills Parking Garage—1961
Photo 1

Corrosion Damage of a Concrete Bridge
Photo 2
Encapsulated Tendon Anchorage Hardware
(Note stressing pocket former to left of wedges)
Photo 3

Encapsulated Tendon Anchorage
(In the upper disassembled unit, note the trumpet grommet to the left of the trumpet)
Photo 4
Schematic Exploded Views of Stressing, Intermediate and Fixed Anchorages

Photo 5
PC Beams with Unbonded Tendons: Analysis in Cracked, Uncracked and Ultimate State

by A.E. Naaman

Synopsis: Assuming the prestressing force remains constant with applied external load, the analysis of beams prestressed with unbonded tendons under service loads, where the section is uncracked, is no different than the analysis of beams with bonded tendons. However, after cracking, significant differences emerge, the most important of which is that the stress in the unbonded tendon is member dependent, that is, it depends on the deformation of the member, while the stress in the bonded tendon is section dependent, that is, it depends on the curvature of the section. This causes the stress in unbonded tendons to be significantly smaller than that for bonded tendons, and that is particularly important at nominal bending resistance.

This paper summarizes the analysis procedure for beams prestressed with unbonded tendons at service and ultimate limit states; it focuses in particular on three problems: 1) how to analyze a beam in the elastic uncracked range of behavior; 2) how to analyze a beam in the elastic cracked range of behavior; and 3) how to analyze a beam at nominal bending resistance.

Keywords: bending analysis; external prestressing; nominal bending resistance; prestressed beams; prestressed slabs; serviceability limit state; ultimate limit state; unbonded tendons
INTRODUCTION

When the bond between the concrete and prestressing tendons is artificially eliminated or minimized, the term "unbonded tendons" is generally used. The analysis and design of concrete beams prestressed with unbonded tendons have evolved for more than half a century to support increasing applications in the field. Initial applications started around 1950 when external prestressing tendons in the form of galvanized strands were used in bridge girders. External prestressing experienced a relatively dormant period until the early 1970’s. Meanwhile, unbonded tendons were extensively used in nuclear power vessels of circular form. However, it is in one- and two-way continuous slab systems that the use of unbonded tendons underwent extensive applications starting in the mid-1950’s. Indeed, prestressed concrete allows longer spans for flat slabs than reinforced concrete and the load balancing characteristic of prestressing offers the advantage of leading to a level slab with zero deflection under permanent load.

The use of unbonded tendons has substantially increased, since their initial introduction, and extends to external prestressing and the use of FRP (fiber reinforced polymeric) reinforcements. Unlike for slab systems, external prestressing implies the use of prestressing tendons outside the concrete section of a structural concrete member. These tendons are primarily designed to ensure horizontal prestressing of the structure and generally represent only a portion of its total reinforcement. The remaining reinforcement may consist of reinforcing steel, prestressing steel, or a combination of them. External prestressing is being increasingly considered in the construction of new concrete structures, particularly bridges, and is a primary method for the rehabilitation and strengthening of old structures.

Recognizing the unusual nature of the behavior of beams and slabs prestressed with unbonded tendons, several studies have provided important clarifications to such behavior \(^1\) to \(^6\). Numerous other studies have addressed various aspects of their analysis and design under service and ultimate loads \(^7\) to \(^16\). The ACI code \(^17\) has several provisions specifically related to the use of unbonded tendons in slab systems, and the ACI-ASCE Committee on Prestressed Concrete regularly updates its document on concrete members prestressed with unbonded tendons \(^18\). A discussion of the ACI code provision related to compression controlled members and unbonded tendons can be found in Refs. \([19\) and \(20]\). The reader is referred to these studies for additional background and information.

Since utilizing unbonded or external tendons poses some uncommon problems in analysis or design, rational methods must be developed to deal with such problems. Rational analytic solutions would allow the accurate determination of stresses and strains at any section along the member and verification of serviceability and strength limit states such as cracking, fatigue, long term deflections and ductility.

The main objective of this paper is to summarize a rational methodology developed by the author over the years for the analysis of beams (and/or slabs) prestressed with
unbonded internal or external tendons. The analysis covers the linear elastic cracked and uncracked ranges of behavior as well as the ultimate limit state in bending. Particular emphasis is given to determining stresses and strains in the constituent materials, steel and concrete, for commonly applied loadings and tendon profiles.

The main feature of the proposed method is to reduce the analysis of beams with unbonded tendons to that of beams with bonded tendons through the use of simple strain reduction coefficients or bond coefficients. These bond coefficients are predetermined for a variety of beams with common tendon profiles and external loadings.

SIGNIFICANCE

The rational analysis of beams prestressed with unbonded tendons is not covered in advanced textbooks on the subject and is often left to research studies where numerical procedures are followed. This paper describes a methodology consistent with the conventional analysis and design procedures for reinforced and prestressed concrete beams at both service and ultimate limit states. It leads to either closed form solutions, or explicit solution equations which can be used directly.

CONCEPT OF STRAIN REDUCTION COEFFICIENT OR BOND COEFFICIENT

In order to reduce the analysis of beams with unbonded tendons to that of beams with bonded tendons, strain reduction coefficients (also called bond reduction coefficients or simply bond coefficients) for the prestressing steel or reinforcement are introduced. These bond reduction coefficients can be determined mathematically for the uncracked state and the cracked state, respectively. The concept of bond reduction coefficient can be extended to the ultimate limit state, although in that case its exact mathematical determination is still subject to various research interpretations.

Reference State: \((F_e + M_D)\)

Linear elastic behavior is assumed. The reference state of stress or strain in the section is that corresponding to the application of \((F_e + M_D)\), where \(F_e\) is the effective value of prestressing force, i.e. \(F_e = A_{ps}f_{pe}\), and \(f_{pe}\) is the effective prestress defined as that obtained after all prestress losses have taken place and in the presence of the dead load moment only. The dead load moment, \(M_D\), is generally taken as the self-weight moment, \(M_G\), but can also be interpreted as the moment due to permanent dead load, such as \((M_G + M_{SD})\), where \(M_{SD}\) is the moment due to superimposed dead load. It is assumed first that the reference state does not lead to cracking in the section (that is, \(M_D < M_{cr}\) ) otherwise the reference state will be a cracked state, and the related analysis described further below would apply.

The corresponding stress diagram along the section is shown in Figs. 1a and 1b for different values of moments. Thus, for \(M = M_D\):

\[
f_p = f_{pe}
\]

\[
F_e = A_{ps}f_{pe}
\]
where \( f_p \) is the stress in the prestressing steel at any loading state, \( f_{pe} \) is the effective stress in the prestressing steel, \( F_e \) is the effective prestressing force, and \( A_{ps} \) is the cross sectional area of the steel.

**Bond Coefficient for the Uncracked State:** \( M_D \leq M \leq M_{cr} \)

This case also corresponds to the case of full prestressing, since the section is assumed uncracked under maximum service moment.

For a moment larger than the dead load moment and smaller that the cracking moment, the following equation can be written (Fig. 1b):

\[
f_p = f_{pe} + \Delta f_p
\]

where \( \Delta f_p \) represents a change in stress in the prestressing tendons due to an increment in bending moment \( \Delta M = (M - M_D) \). Assuming linear elastic behavior:

\[
\Delta f_p = E_p \Delta \varepsilon_p
\]

where \( \Delta \varepsilon_p \) represents the strain change in the prestressing steel in the section considered. For bonded tendons, \( \Delta \varepsilon_p \) is also equal to the strain change in the concrete at the level of the steel in that section, \( \Delta \varepsilon_{cp} \); thus, for any section located a distance \( x \) from the support:

\[
[\Delta \varepsilon_p(x)]_{\text{bonded}} = \Delta \varepsilon_{cp}(x)
\]

If the section of maximum bending moment is being analyzed, then \( \Delta \varepsilon_{cp} \) is also the strain change in the critical design section (here the midspan section), \( (\Delta \varepsilon_{cp})_{\text{max}} \), that is:

\[
[\Delta \varepsilon_p(x)]_{\text{bonded-max}} = [\Delta \varepsilon_{cp}]_{\text{max}}
\]

where the subscript "max" refers to the section of maximum moment. It can be shown that:

\[
(\Delta \varepsilon_{cp})_{\text{max}} = \frac{(M_{\text{max}} - M_D)}{E_c I_g} (e_o)_{\text{max}} = \frac{\Delta M_{\text{max}}}{E_c I_g} (e_o)_{\text{max}}
\]

where \( E_c \) is the modulus of elasticity of concrete, \( I_g \) is the section gross moment of inertia which can be also taken as the transformed moment of inertia of the uncracked section, \( \Delta M_{\text{max}} \) is the change in moment in the section of maximum moment, and \( (e_o)_{\text{max}} \) is the eccentricity of the tendons at the section of maximum moment.

For unbonded tendons the change in stress in the tendons at any section, \( x \), is assumed given by:

\[
(\Delta \varepsilon_p)_{\text{unbonded}} = (\Delta \varepsilon_p)_{\text{average}}
\]

where \( (\Delta \varepsilon_p)_{\text{average}} \) is the average strain increase in the unbonded tendon over the length of the member. Thus, in order to determine the stress increment in a tendon, we need to determine the average strain increment over the span.

Let us define the strain reduction coefficient or bond coefficient \( \Omega \) as follows:

\[
\Omega = \frac{(\Delta \varepsilon_p)_{\text{unbonded-max}}}{(\Delta \varepsilon_p)_{\text{bonded-max}}} = \frac{(\Delta \varepsilon_p)_{\text{unbonded-average}}}{(\Delta \varepsilon_p)_{\text{bonded-average}}} = \frac{(\Delta \varepsilon_{cp})_{\text{unbonded-average}}}{(\Delta \varepsilon_{cp})_{\text{bonded-max}}}
\]

**Naaman**
where the subscript "max" implies the midspan section or the section of maximum moment. The subscript "p" refers to the prestressing tendon while the subscript "cp" refers to the concrete fiber at the level of the tendon. Equation (9) is valid for fully unbonded tendons with zero frictional coefficient. Note that for bonded tendons, $\Omega = 1$, as indicated by the first term of the left hand side of Eq. (9). For fully unbonded tendons, $\Omega$ varies between zero and one and can be calculated from Eq. (9); thus:

$$
(\Delta \varepsilon_p)_{\text{unbonded-average}} = (\Delta \varepsilon_{cp})_{\text{unbonded-average}} = \frac{1}{\ell} \int_0^\ell \Delta \varepsilon_{cp}(x) \, dx
$$

(10)

where at any section $x$ along the span

$$
\Delta \varepsilon_{cp}(x) = \frac{M(x) - M_D(x)}{E_c I_g} e_o(x) = \frac{\Delta M(x)}{E_c I_g} e_o(x)
$$

(11)

in which $\Delta M(x)$ is the moment in excess of reference state moment (dead load moment) at section $x$. For the section of maximum moment, Eq. (11) leads to:

$$
(\Delta \varepsilon_{cp})_{\text{bonded-max}} = \frac{M(x) - M_D(x)}{E_c I_g} e_o(x) = \frac{\Delta M_{\text{max}}}{E_c I_g} (e_o)_{\text{max}}
$$

(12)

From Eqs. (10 to 12) a value of $\Omega$ can be generally obtained.

**General analytical expression and numerical derivation** -- For the numerical derivations for use in common design, the following assumptions are made:

- Simply supported beams with constant $E_c I_g$ throughout their length
- Symmetrical loading and tendon profile with respect to midspan
- Linear elastic materials in the range of behavior considered
- Linear strain distribution along the concrete section
- Second order effects, if any, for external tendons are negligible.

Thus the critical design section is the midspan section at which the maximum eccentricity of the tendons is assigned. In that case it can be shown that $\Omega$ (Eq. 9 to 12) can be calculated in the most general case from the following equation:

$$
\Omega = \frac{2}{\ell \times \Delta M_{\text{max}} \times (e_o)_{\text{max}}} \int_0^{\ell/2} \Delta M(x) \times e_o(x) \, dx
$$

(13)

Eq. (13) implies, as is mostly the case for most loading and tendon profiles, that the moment and tendon eccentricity at any section, $x$, can be determined respectively from their value at the section of maximum moment multiplied by a function of $x$. Applying the above equation to common loading cases and tendon profiles leads to the values of $\Omega$ listed in the third column of Table 1.

**Bond Coefficient for the Elastic Cracked State:** $M \geq M_{cr} \geq M_D$

The section stress and strain diagrams are illustrated in Fig. 2. The definition of bond coefficient $\Omega$ (described above) can be extended to the cracked state of behavior using the same basic definition given by Eq. (9). It will be assumed that once the applied moment exceeds the cracking moment, a crack will appear in the midspan region. For the purpose of analysis, let us assume that only one crack will form at the section of
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maximum moment and let us analyze the beam assuming that it is divided into two parts, one uncracked segment with moment of inertia \( I_{\text{transformed-uncracked}} \) (or, as a first approximation, \( I_g \)) and one cracked segment with moment of inertia \( I_{cr} \) (that is, \( I_{\text{transformed-cracked}} \)) (Fig. 3b). Assume that the cracked portion of beam has a length \( \ell_c \), while the uncracked portion has a length \( (\ell - \ell_c) \). In a manner similar to what was done in the previous section, one can define a bond coefficient \( \Omega_{cr} \) which represents the ratio of average strain change in the unbonded tendon over the entire span, to the strain change in the concrete at the level of the steel at the section of maximum moment, \((\Delta\varepsilon_{cp})_{\text{max}}\). However, \((\Delta\varepsilon_{cp})_{\text{max}}\) is also equal to the strain change in the equivalent bonded tendon assuming a cracked section. Thus:

\[
\Omega_{cr} = \frac{(\Delta\varepsilon_p)_{\text{unbonded-max}}}{(\Delta\varepsilon_p)_{\text{bonded-max}}} = \frac{(\Delta\varepsilon_p)_{\text{unbonded-average}}}{(\Delta\varepsilon_p)_{\text{bonded-max}}} = \frac{(\Delta\varepsilon_{cp})_{\text{unbonded-average}}}{(\Delta\varepsilon_{cp})_{\text{bonded-max}}} \quad (14)
\]

\[
(\Delta\varepsilon_p)_{\text{unbonded-average}} = (\Delta\varepsilon_{cp})_{\text{unbonded-average}} = \frac{1}{\ell} \int_0^\ell \Delta\varepsilon_{cp}(x)dx \quad (15)
\]

For a symmetrical loading and symmetrical tendon profile, Eq. (15) can be written as:

\[
(\Delta\varepsilon_p)_{\text{unbonded-average}} = \frac{2}{\ell} \left( \frac{\ell - \ell_c}{2} \Delta\varepsilon_{cp}(x)dx \right) + \frac{2}{\ell} \left( \frac{\ell/2}{(\ell - \ell_c)/2} \Delta\varepsilon_{cp}(x)dx \right) \quad (16)
\]

Along the uncracked portion of the beam, we have:

\[
\Delta\varepsilon_{cp}(x) = \frac{\Delta M(x)E_o(x)}{E_c I_g} \quad (17)
\]

where \( I_g \) is the gross moment of inertia of the uncracked section (which can also be taken as the transformed moment of inertia of the uncracked section).

At any section along the cracked portion of the beam, we have:

\[
\Delta\varepsilon_{cp}(x) = \frac{M_j(x)E_o-cr(x)}{E_c \times (I_{cr})_j} - \frac{M_i(x)E_o-cr(x)}{E_c \times (I_{cr})_i} \quad (18)
\]

Eq. (18) simulates Eq. (17); however, \( I_g \) is replaced by the moment of inertia of the cracked section \( I_{cr} \). \( M_i \) and \( M_j \) are two consecutive values of moments at section \( x \); they include the contribution of prestressing moment. \( \Delta M \) in Eq. (17) is equivalent to \( \Delta M = M_j - M_i \). For each applied moment in the cracked state, the values of neutral axis \( c \), the corresponding moment of inertia \( I_{cr} \), and the strain \( \varepsilon_{cp}(x) \) can be calculated from the combined effects of external moment applied, the prestressing force, and its eccentricity [Refs. 21, 22, 23]. Different values of \( c \), \( I_{cr} \), and \( \varepsilon_{cp}(x) \) are obtained for different values of moment allowing to compute \( \Delta\varepsilon_{cp}(x) \). Note that \( e_{o-cr}(x) \) represents the distance from the centroidal axis of bending of the cracked section to the centroid of the prestressing steel and is different from \( e_o(x) \) which is taken with respect to the centroidal axis of the uncracked section.
If we assume \( c \) to be small (such as a crack width (Fig. 3b) or a plastic hinge length), then the steel eccentricity and the moment along \( c \) can be assumed uniform; similarly \( I_{cr} \) can be assumed constant along \( c \). Hence, the change in strain in the concrete at the level of the steel at any section \( x \) along \( c \) can be assumed approximately equal to the change in strain at the section of maximum moment, \( (\Delta \varepsilon_{cp})_{\text{max}} \). Thus for a given moment leading to cracking:

\[
\int_{-c/2}^{c/2} (\Delta \varepsilon_{cp}(x)) dx = (\Delta \varepsilon_{cp})_{\text{max}}
\]  

Replacing Eqs. (17 and 18) in Eq. (16) and solving leads to the value of \((\Delta \varepsilon_{p})_{\text{unbonded-average}}\). Using then Eqs. (14 and 19) leads to the value of \( \Omega_{cr} \). It can be shown that, for symmetrical loading and tendon profile, the value of \( \Omega_{cr} \) can be obtained in the most general manner from the following equation:

\[
\Omega_{cr} = \frac{\Omega I_{cr}}{I_g} + \left( \frac{1}{I_g} \right) \int_{0}^{\ell/2} \frac{\Delta \varepsilon_{cp}(x) dx}{(\Delta \varepsilon_{cp})_{\text{max}}} \ll (19)
\]

in which the value of the fractional expression under the integral is one. Eq. (20) leads to the following general solution:

\[
\Omega_{cr} \approx \frac{\Omega I_{cr}}{I_g} + \left( \frac{1}{I_g} \right) \frac{\ell}{\ell_c} \ll (20)
\]

Whether \( \ell_c \) is interpreted as a crack width at the critical section or the length of beam segment where cracking would occur, the ratio \( \ell_c / \ell \) can be generally assumed to be very small. Moreover, the ratio \( I_{cr} / I_g \) is smaller than one (generally between 0.2 and 0.5); thus the second term on the right side of Eq. (21) can be neglected in most situations leading to the following simple result which can be used for practical design:

\[
\Omega_{cr} \approx \frac{\Omega I_{cr}}{I_g} \ll (21)
\]

The values of \( \Omega_{cr} \) are listed in the fourth column of Table 1 for common loading and tendon profiles. In Eqs. (21 and 22), \( I_{cr} \) is the moment of inertia of the cracked section of maximum moment for the applied service loading leading to cracking.

**Bond Coefficient for the Ultimate Limit State in Bending:** \( M_u = M_n \)

If a bond coefficient is defined for the ultimate limit state, it could be assumed, by inference at first, to take the same form as the bond coefficient \( \Omega_{cr} \) defined for the cracked state (Eq. 21). Thus, hypothetically:

\[
\Omega_u \approx \frac{\Omega (I_{cr})_u}{I_g} + \left( \frac{1}{I_g} \right) \frac{(\ell_c)_u}{\ell} \ll (23)
\]
in which \((I_{cr})_u\) is the moment of inertia of the cracked section at ultimate (i.e., at nominal bending resistance), and \((\ell_c)_u\) is the length of segment assumed cracked at ultimate. \((\ell_c)_u\) can also be interpreted essentially as the length of plastic hinge at the section of maximum moment at ultimate (Fig. 3c). The author has attempted in the past to derive numerical values for the bond coefficient \(\Omega_u\) for various loading and tendon profiles, based on Eq. (23). No simple answer could be found and Eq. (23) could not be validated. However, it does point out to a number of influencing parameters. For instance \(\Omega\) depends on the loading and tendon profile, \((I_{cr})_u / I_g\) illustrates reduction of stiffness, and \((\ell_c)_u / \ell\) is representative of a plastic hinge length.

Note that the basic definition of \(\Omega_u\) is same as that stipulated for \(\Omega\) and \(\Omega_{cr}\) in Eqs. (6 and 14), that is, the ratio of strain increment in the unbonded tendon to that of the equivalent bonded tendon at the section of maximum moment. In an extensive evaluation of the test results of more than 143 beams taken from different investigations, Naaman and Alkhairi\(^\text{12-13}\) used a regression analysis to derive values of the bond coefficient \(\Omega_u\). Based on their findings, they recommended the following values of \(\Omega_u\) for design:

\[
\Omega_u \approx \frac{3}{L / d_p} \quad \text{for uniform loading or third point loading} \quad (24)
\]

\[
\Omega_u \approx \frac{1.5}{L / d_p} \quad \text{for single point loading at midspan} \quad (25)
\]

in which \(L\) is the span length and \(d_p\) is the depth from the extreme compression fiber to the centroid of the prestressing tendons at nominal bending resistance. These values are shown in the last columns of Table 1 for easy comparison with the values of \(\Omega\) and \(\Omega_{cr}\). Assuming \(\Omega_u\) is given, the stress in the unbonded prestressing tendons can then be easily determined from a pseudo-strain compatibility based on a deflection compatibility (see section Analysis at Ultimate below and Eq. 55). The corresponding computation of nominal bending resistance is covered in details in Refs. 16 and 20.

**FLEXURAL ANALYSIS: APPROACH AND ASSUMPTIONS**

A schematic representation of the moment deflection relationship of a beam prestressed or partially prestressed with unbonded internal or external tendons is shown in Fig. 4. This curve could also represent the moment curvature relation of a beam section. The curve can be conceptually divided into several parts: part \(AB\) corresponds to the linear elastic uncracked range of behavior; part \(BC\) illustrates the change in deflection at onset of cracking; part \(CD\) represents the linear elastic cracked range; part \(DE\) corresponds to the cracked nonlinear range of behavior; point \(E\) represents the nominal or ultimate resistance; and point \(F\) the failure point. The curve of Fig. 4 describes the range of possible behavior. In theory a sudden change in deflection occurs at the onset of cracking (segment \(BC\)); however the real behavior shows a more gradual change in slope from \(AB\) to \(DE\).
Actual design of beams with unbonded internal or external tendons may or may not allow cracking under service loads. However, since loads are random in nature and overloads are quite common, the cracking state should be considered in any comprehensive evaluation. This is also the case should repair-replacement of some tendons be needed after corrosion or other damage. Moreover, cracking is allowed by the ACI code in partially prestressed members using bonded tendons, and in one-way slabs with unbonded tendons provided sufficient amount of non-prestressed conventional steel is added. In U.S. practice, both service load design (i.e., working stress design or allowable stress design) and ultimate strength design (or load and resistance factor design) are required for prestressed concrete structures. Service load design implies linear elastic behavior in the uncracked and cracked range.

Assumptions

The assumptions for the analysis are as follows:

1. Both steel and concrete are linear elastic in the range of stresses considered; stresses and strains are directly proportional as per Hooke’s law
2. Plane sections remain plane under bending
3. No bond exists between prestressing steel and concrete
4. Concrete does not withstand tensile stresses when cracked section analysis is considered
5. Second order effects for external tendons, if any, are negligible

Based on these assumptions, a mathematical solution can be derived for the analysis of the two cases of uncracked and cracked concrete sections. It is an extension of a similar analysis derived for partially prestressed beams with bonded tendons \(^{21-22}\), with a modification to account for the bond coefficients. The main feature of the proposed method is that the analysis of beams with unbonded tendons is reduced to that of beams with bonded tendons when the bond coefficients are taken equal to unity.

**LINEAR ELASTIC ANALYSIS IN THE UNCRACKED STATE (\(M \leq M_{cr}\))**

This state is represented by segment AB in Fig. 4. The material components, steel and concrete, are assumed to work in their linear elastic range of behavior. However, no bond exists between prestressing tendons and concrete. The stress and strain diagrams along the section are shown in Fig. 1.

It is assumed that the critical design section is the midspan section at which the maximum eccentricity of the tendons is assigned. Simply supported beams are considered first.

The methodology described below, consists of reducing the analysis with unbonded tendons to that with bonded tendons through the use of the bond coefficient \(\Omega\). The coefficient \(\Omega\) depends only on the steel profile and type of loading, and needs to be determined only once for common loading and tendon configurations (see Table 1).

While in common design the effective prestressing force (i.e., the force remaining after all prestress losses have occurred) is assumed to remain constant under different levels of service loads, here the prestressing force \(F\) is assumed to vary with the applied
load. The corresponding stress in the prestressing steel is defined as \( f_p \). Thus, \( f_p \) is a variable and refers to the stress in the prestressing steel for any loading considered, thus:

\[
F = A_{ps} f_p
\]  

(26)

The proposed methodology generates equations to determine the stresses and stress changes in the component materials for any moment larger than the dead load moment, \( M_D \), and smaller than the cracking moment, \( M_{cr} \). In particular, solution equations providing the stress in the prestressing steel, the stress in the non-prestressed steel, the stress in the concrete extreme compression fiber, and the cracking moment are developed and summarized in Table 2. Note that for concrete, compression is considered positive and tension negative; while for steel, compression is negative and tension is positive.

Reference State: \( (F_e + M_D) \)

The reference state was described above for the derivation of the bond coefficient, \( \Omega \). For the reference state, the following relations can be derived (see also Fig. 1):

\[
f_p = f_{pe}
\]  

(27)

\[
\varepsilon_{pe} = \frac{f_{pe}}{E_{ps}}
\]  

(28)

\[
\varepsilon_{ce} = \frac{1}{E_c} \left[ \frac{A_{ps} f_{pe}}{I} \left( r^2 + e_o^2 \right) - \frac{M_D e_o}{I} \right]
\]  

(29)

\[
f_{cps} = f_{pe} A_{ps} \left[ \frac{1}{A_c} + \frac{e_o^2}{I} \right] - \frac{M_D e_o}{I}
\]  

(30)

\[
f_{cns} = f_{pe} A_{ps} \left[ \frac{1}{A_c} + \frac{e_o (d_s - y_t)}{I} - \frac{M_D (d_s - y_t)}{I} \right]
\]  

(31)

\[
f_s = -\frac{E_s}{E_c} f_{cns} = -\frac{E_s}{E_c} \left\{ f_{pe} A_{ps} \left[ \frac{1}{A_c} + \frac{e_o (d_s - y_t)}{I} \right] - \frac{M_D (d_s - y_t)}{I} \right\}
\]  

(32)

\[
f'_s = \frac{E'_s}{E_c} \left\{ f_{pe} A_{ps} \left[ \frac{1}{A_c} - \frac{e_o (y_t - d'_s)}{I} \right] + \frac{M_D (y_t - d'_s)}{I} \right\}
\]  

(33)

\[
f_{ct} = -\frac{f_{pe} A_{ps}}{A_c} \left( \frac{1 - e_o}{k_b} \right) + \frac{M_D y_t}{I}
\]  

(34)

where \( f_{cps} \) and \( f_{cns} \) are the stresses in the concrete, respectively, at the level of the prestressed and the non-prestressed steel, and \( f_{ct} \) is the stress in the concrete extreme compression fiber (Fig. 1).

Any moment larger than the dead load moment will lead to an increase in the value of \( F \) (or equivalently \( f_p \)). The increase in prestressing steel stress, \( \Delta f_p \), will affect the
value of the cracking moment and vice versa. Once the stress in the prestressing steel \( f_p \) is determined, other stresses in the section can be easily calculated using the corresponding value of the prestressing force.

**Uncracked Section for \((F + M)\) with \(M_D \leq M \leq M_{cr}\)** (Segment AB of Fig. 4)

For a moment larger than the dead load moment, the stress in the prestressing tendons increases leading to the following expression:

\[
fp = fp_e + \Delta fp
\]

(35)

where \( \Delta fp \) represents a change in stress in the prestressing tendons.

The use of the coefficient \( \Omega \) defined earlier (Eq. 13) can be integrated in the analysis of the uncracked section to generate expressions for the stresses and strains in the steel and concrete at the critical section (i.e. section of maximum moment and maximum eccentricity) for any moment between the dead load moment and the cracking moment. The cracking moment can also be determined assuming that the prestressing force or the stress in the steel varies with the applied moment. Let us illustrate how to determine the change in stress in the prestressing steel.

Referring to Eq. (4) the stress change for unbonded tendons is given by:

\[
\Delta fp = E_{ps} (\Delta \epsilon_p)_{\text{unbonded-average}}
\]

(36)

and from Eq. (10) where \((\Delta \epsilon_p)_{\text{unbonded-average}} = (\Delta \epsilon_{cps})_{\text{unbonded-average}}\), we have:

\[
(\Delta \epsilon_p)_{\text{unbonded-average}} = \Omega (\Delta \epsilon_{cps})_{\text{max}}
\]

(37)

The strain change in the concrete at the level of the steel at the section of maximum moment is related to the stress change in the concrete at the same level, that is:

\[
(\Delta \epsilon_{cps})_{\text{max}} = \frac{(\Delta f_{cps})_{\text{max}}}{E_c}
\]

(38)

The stress change in the concrete is the difference in stress between the reference state (dead load plus effective prestress) and the current state (moment \(M < M_{cr}\) and actual prestress):

For the reference state \((F_e + M_D)\):

\[
(f_{cps})_{\text{max}} = \frac{f_{pe} A_{ps} (e_o^2 - r^2) - M_D e_o}{I_g}
\]

(39)

where \(r\) is the radius of giration of the section and \(I_g\) is the gross moment of inertia which replaces as a first approximation the transformed moment of inertia of the uncracked section.
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For the actual state \((F + M)\):

\[
(f_{cps})_{\text{max}} = \frac{(f_{pe} + \Delta f_p)A_{ps}(e_o^2 - r^2) - Me_o}{I_g} \tag{40}
\]

Subtracting Eq. (39) from Eq. (40) leads to \((\Delta f_{cps})_{\text{max}}\); thus \((\Delta e_{cps})_{\text{max}}\) can be obtained from Eq. (38), and \(\Delta f_p\) from Eqs. (36 and 37). Solving the resulting equation for \(\Delta f_p\) leads to the following expression:

\[
\Delta f_p = \frac{\Omega(M - M_D)e_o}{I_g \frac{E_c}{E_{ps}} + \Omega A_{ps}(e_o^2 + r^2)} \tag{41}
\]

where \(\Delta f_p\) is the stress change in the prestressing steel with respect to the reference state for a moment larger than the dead load moment and smaller than the cracking moment. Given \(\Delta f_p\), the corresponding stress in the concrete and the reinforcing steel can be easily determined as well as the cracking moment. The solution procedure is similar to that described in Ref. [21], and the solution equations are given in Table 2; they apply to any state of loading between the reference state and the cracking state (Fig. 1) assuming linear elastic uncracked section (or member) behavior. It can be observed that for \(\Omega = 1\), these equations revert to the equations given in Refs. [21 and 22] for partially restressed beams with bonded tendons. Note that the cracking moment, assuming \(f_p\) variable, is equal to the sum of cracking moment assuming \(f_p = f_{pe}\) (i.e., for the reference state) plus an increment of moment \(\Delta M_{cr}\). The cracking moment corresponding to the reference state, here defined as \((M_{cr})_c\) is the same as the usual cracking moment obtained assuming \(F\) remains constant for any applied service moment. Finally, note that the moment leading to decompression in the concrete is obtained from the cracking moment in which the modulus of rupture of concrete, \(f_r\), is taken equal to zero.

LINEAR ELASTIC ANALYSIS IN THE CRACKED STATE, \(M \geq M_{cr}\)

This part of the analysis covers segment CD of the moment deflection curve described in Fig. 4.

If the applied service moment exceeds the cracking moment, a crack will appear in the midspan region (Fig. 3b). A typical beam cross section and the corresponding stress and strain diagrams are shown in Fig. 2.

The mathematical equations expressing equilibrium, compatibility and stress-strain relationships assuming that the constituent materials remain in the linear elastic range of behavior, can be set similarly to the case of the analysis of a cracked section containing bonded prestressing steel and conventional reinforcing bars. However, the strain compatibility equations related to the prestressing steel will contain the bond coefficient \(\Omega_{cr}\) described earlier (Eq. 20 to 22). These equations are given below:
Force Equilibrium:

$$A_{ps} f_p + A_s f_s = b f_{ct} \frac{c}{2} - f_{ct} \left( \frac{b - b_w}{2} \right) \frac{(c - h_f)^2}{c} - A'_s f'_s$$

(42)

where the notation is standard. Note that compression in the steel is assumed negative, thus $f'_s$ is negative. The above equation applies to rectangular and T-sections. For a rectangular section use $b = b_w$.

Moment Equilibrium:

Taking the moments with respect to centroid of compressive forces:

$$A_{ps} f_p (d_p - \overline{x}) + A_s f_s (d_s - \overline{x}) = M$$

(43)

where $\overline{x}$ is the distance from the extreme compression fiber to the centroid of the compressive forces.

Centroid of Compressive Forces $\overline{x}$ (Fig. 2a):

$$\overline{x} = \frac{1}{3} \left[ 6A'_s f'_s d'_s c + f_{ct} \left[ b c^3 - (b - b_w)(c + 2h_f)(c - h_f)^2 \right] \right]$$

(44)

Note that $\overline{x}$ is not needed for the solution since the moment can also be taken with respect to the extreme compression fiber; however, it is introduced here for convenience and because the centroid of compression may be needed for other purposes.

Strain Compatibility and Stress-Strain Relations:

$$\varepsilon_{ct} = \frac{f_{ct}}{E_c}$$

(45)

$$\varepsilon_s = \frac{f_s}{E_s} = \frac{f_{ct}}{E_c} \left[ \frac{d_c - c}{c} \right]$$

(46)

$$\varepsilon_p = \varepsilon_{pe} + \Delta \varepsilon_p = \varepsilon_{pe} + \Omega_{cr} (\varepsilon_{ce} + \varepsilon_{cps}) = \varepsilon_{pe} + \Omega_{cr} \left[ \varepsilon_{ce} + \frac{f_{ct}}{E_c} \left( \frac{d_p - c}{c} \right) \right]$$

(47)

$$\varepsilon'_s = \frac{f'_s}{E'_s} = \frac{f_{ct}}{E_c} \left( \frac{c - d_s}{c} \right)$$

(48)

$$f_s = E_s \varepsilon_s = \frac{E_s}{E_c} f_{ct} \left( \frac{d_s - c}{c} \right)$$

(49)

$$f_p = E_{ps} \varepsilon_p = E_{ps} \varepsilon_{pe} + \Omega_{cr} E_{ps} \varepsilon_{ce} + \Omega_{cr} \frac{E_{ps}}{E_c} f_{ct} \left( \frac{d_p - c}{c} \right)$$

(50)
Equations (42) to (51) have 10 unknowns, namely: \( c, f_{ct}, f_s, f_p, f_s', \varepsilon_{ct}, \varepsilon_s, \varepsilon_p, \varepsilon_s', \) and \( \bar{x} \). Moreover, the six equations not dealing with strain values have only six unknowns: \( c, f_{ct}, f_s, f_p, f_s', \bar{x} \). It is not possible to obtain a closed-form solution for each variable; however, the manipulation of these equations leads to a single most general cubic equation in \( "c" \) where \( c \) is the distance from the extreme compression fiber to the zero stress point (Fig. 3). Once \( c \) is determined other stresses along the cross section can be calculated.

It is convenient sometimes to use in the above equations or in their solutions, the moduli ratios as defined below:

\[
\frac{n_p}{P} = \frac{E_p}{E_c} \quad (52) \\
\frac{n_s}{S} = \frac{E_s}{E_c} \quad (53) \\
\frac{n'_s}{S'} = \frac{E'_s}{E_c} \quad (54)
\]

The general solution equations are given in Table 3. The first equation of Table 3 (Eq. 66) is a cubic equation in \( c \) where \( c \) is the depth to the neutral axis (zero stress point) of the elastic cracked section. As \( \Omega_{cr} \) must be estimated from Eq. (22) which contains the term \( I_{cr}/I_g \) and, since \( I_{cr}/I_g \) depends on \( c \), some iteration in the solution of first equation of Table 3 may be needed.

The following computational steps can be followed: assume a value of \( I_{cr}/I_g \) (say 0.3), determine \( c \) from Eq. (66), compute corresponding \( I_{cr} \), and check if the assumed \( I_{cr}/I_g \) is acceptable. Otherwise, iterate. Once an acceptable value of \( c \) is obtained, the corresponding stresses in the constituent materials, steel and concrete, can be determined from the equations of Table 3. In particular the stress in the unbonded prestressing steel can be calculated for any applied moment larger than the cracking moment and smaller than the moment leading to inelastic behavior in one of the components materials. Note that the solution equations given in Table 3 cover prestressed and partially prestressed rectangular and T sections with and without compressive reinforcement. For rectangular sections use \( b = b_w \). When the value of \( \Omega_{cr} \) is taken equal to 1, these equations revert to the case of beams prestressed with bonded tendons and become identical to the solution equations derived earlier in Ref. [21].

**ANALYSIS AT ULTIMATE**

The analysis of beams prestressed or partially prestressed with unbonded tendons at ultimate (or nominal bending resistance) can be approached in several ways each having a different level of complexity. In the simplest approach a gross prediction equation for...
the stress in the prestressing steel is used simultaneously with the equations of force and moment equilibrium. A more refined procedure detailed at length in Refs. [12, 13 and 16] utilizes a deflection compatibility analysis reduced to a pseudo-strain compatibility through the use of a strain reduction coefficient or bond coefficient, $\Omega_u$. Assuming the values of $\Omega_u$ are given such as in Eqs. (24, 25) and Table 1, the stress in the unbonded prestressing steel at ultimate can be predicted from the following equation

$$f_{ps} = f_{pe} + \Omega_u E_p \varepsilon_{cu} \left( \frac{d_p}{c} - 1 \right) \frac{L_1}{L_2} \leq 0.80 f_{pu}$$

where:

- $f_{pe}$ = effective prestress in the prestressed reinforcement
- $E_p$ = elastic modulus of prestressed reinforcement
- $\varepsilon_{cu}$ = assumed failure strain of concrete in compression = 0.003
- $d_p$ = distance from extreme compression fiber to centroid of prestressed reinforcement
- $c$ = depth of neutral axis at nominal bending resistance
- $L$ = span length for which computation is carried out. At intermediate supports of continuous members $L$ can be taken as the average of the two spans on either side of the support.
- $L_1$ = sum of lengths of spans loaded by live load and containing the same tendon
- $L_2$ = total length of tendon between anchorages
- $\Omega_u$ = $3/(L/d_p)$ for uniform or third point loading
- $\Omega_u$ = $1.5/(L/d_p)$ for one point midspan loading
- $f_{pu}$ = specified tensile strength of prestressed tendons

Note that the value of $\Omega_u$ here is not related to that in Eq. (23) but is explained in relation to Eqs. (24) and (25). The value of depth of neutral axis, $c$, at ultimate accounts for the presence of non-prestressed reinforcement and satisfies equilibrium at ultimate. The equation of force equilibrium in the section and the prediction equation of $f_{ps}$ must be solved simultaneously for $c$ and $f_{ps}$.

An extensive discussion, validation, and condition of application Eq. (55) as well as numerous examples of computation of nominal bending resistance can be found in Refs. [16 and 20].

CONCLUDING REMARKS

The methodology described in this paper allows for a rational yet simplified analysis of beams prestressed or partially prestressed with unbonded tendons in the linear elastic cracked or uncracked range of behavior, and at nominal bending resistance. It provides a link between the case of bonded tendons and the case of unbonded tendons through the
use of strain reduction coefficients or bond coefficients. The bond coefficients for the elastic uncracked and cracked behavior, $\Omega$ and $\Omega_{cr}$, are computed only once for common combinations of loadings and tendon profiles (Table 1). Numerical values of bond coefficients for the ultimate limit states, $\Omega_u$, are provided based on a regression analysis and should be viewed as approximate. Substantial additional research may be needed to fine-tune the values of $\Omega_u$. When the bond coefficients are taken equal to unity, the solutions provided revert to the case of prestressed and partially prestressed beams with bonded tendons. This offers the advantage of having a generalized solution that is equally valid for bonded and unbonded tendons.

DEDICATION

This paper is dedicated to Ned Burns in recognition of his numerous contributions to the understanding, design, analysis, and education related to concrete members, particularly slabs, prestressed with unbonded tendons.

ACKNOWLEDGMENTS

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REFERENCES

17. ACI Committee 318, *Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02)*, American Concrete Institute, Farmington Hills, Mich., 2002, 443 pp.
18. Joint ACI-ASCE Committee 423, *Recommendations for Concrete Members Prestressed with Unbonded Tendons (ACI 423.3R-02)*, American Concrete Institute, 2003, 19pp.
Table 1 Expressions of the bond coefficient for the uncracked elastic, cracked elastic, and ultimate state.

<table>
<thead>
<tr>
<th>Loading Type</th>
<th>Tendon Profile</th>
<th>Uncracked Member</th>
<th>Cracked Member</th>
<th>At Nominal Bending</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform Load</td>
<td>Straight</td>
<td>( \Omega = \frac{2}{3} )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{3}{L/d_p} )</td>
</tr>
<tr>
<td>Uniform Load</td>
<td>Single draping at midspan</td>
<td>( \Omega = \frac{5}{12} + \frac{1}{4} e_s )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{3}{L/d_p} )</td>
</tr>
<tr>
<td>Uniform Load</td>
<td>Parabolic</td>
<td>( \Omega = \frac{8}{15} + \frac{2}{15} e_m )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{3}{L/d_p} )</td>
</tr>
<tr>
<td>Third-Point Loads</td>
<td>Straight</td>
<td>( \Omega = \frac{2}{3} )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{3}{L/d_p} )</td>
</tr>
<tr>
<td>Third-Point Loads</td>
<td>Single draping at midspan</td>
<td>( \Omega = \frac{23}{81} + \frac{13}{81} e_s )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{3}{L/d_p} )</td>
</tr>
<tr>
<td>Third-Point Loads</td>
<td>Parabolic</td>
<td>( \Omega = \frac{44}{81} + \frac{10}{81} e_m )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{3}{L/d_p} )</td>
</tr>
<tr>
<td>Single Midspan Load</td>
<td>Straight</td>
<td>( \Omega = \frac{1}{2} )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{1.5}{L/d_p} )</td>
</tr>
<tr>
<td>Single Midspan Load</td>
<td>Single draping at midspan</td>
<td>( \Omega = \frac{1}{3} + \frac{1}{6} e_m )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{1.5}{L/d_p} )</td>
</tr>
<tr>
<td>Single Midspan Load</td>
<td>Parabolic</td>
<td>( \Omega = \frac{5}{12} + \frac{1}{4} e_m )</td>
<td>( \Omega_{cr} \approx \frac{I_{cr}}{I_g} )</td>
<td>( \Omega_u \approx \frac{1.5}{L/d_p} )</td>
</tr>
</tbody>
</table>

Note: \( e_s \) is the eccentricity at supports and \( e_m \) is the eccentricity at midspan. The eccentricity is assumed positive below the neutral axis and negative above the neutral axis.
Table 2 Summary of solution equations for the uncracked section, $M_D \leq M \leq M_{cr}$.

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Solution Equation</th>
<th>Equation Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>$(F + M_D)$</td>
<td>$f_p = f_{pe}$</td>
<td>(1)</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_{pe} = \frac{f_{pe}}{E_p}$</td>
<td>(2)</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_{ce} = \frac{1}{E_c} \left[ \frac{A_p}{f_{pe}} \left( \varepsilon_0^2 + \varepsilon_0^2 - \frac{M_D}{I} \right) \right]$</td>
<td>(3)</td>
</tr>
<tr>
<td>$(F + M)$</td>
<td>$f_p = f_{pe} + \frac{\Omega (M - M_D) \varepsilon_0}{E_p + \Omega A_p \left( \varepsilon_0^2 + \varepsilon_0^2 \right)}$</td>
<td>(4)</td>
</tr>
<tr>
<td>$M_D &lt; M &lt; M_{cr}$</td>
<td>$f_{cr} = f_p \frac{A_p}{A_c} \left[ 1 + \frac{\varepsilon_0 (d_z - y_f)}{r^2} \right] \frac{M (d_z - y_f)}{I}$</td>
<td>(5)</td>
</tr>
<tr>
<td></td>
<td>$f_s = \frac{E_p}{E_c} f_{cs}$</td>
<td>(6)</td>
</tr>
<tr>
<td></td>
<td>$f_{cr} = f_p \frac{A_p}{A_c} \left[ 1 - \frac{\varepsilon_0}{k_b} \right] + \frac{M y_f}{I}$</td>
<td>(7)</td>
</tr>
<tr>
<td>$(F + M_{cr})$</td>
<td>$M_{cr} = (M_{cr})<em>c + \Delta M</em>{cr}$</td>
<td>(8)</td>
</tr>
<tr>
<td></td>
<td>$(M_{cr})<em>c = A_p f</em>{pe} \left( \varepsilon_0 + \frac{Z_b}{A_c} \right) + 7.5 Z_b \sqrt{f_c}$</td>
<td>(9)</td>
</tr>
<tr>
<td></td>
<td>$\Delta M_{cr} = \frac{A_p \varepsilon_0 \left( \varepsilon_0 + \frac{Z_b}{A_c} \right) \left[ (M_{cr})_c - M_D \right]}{f_p E_p \frac{1}{\Omega} + A_p \left( \varepsilon_0^2 - \frac{Z_b}{A_c} \right)^2}$</td>
<td>(10)</td>
</tr>
</tbody>
</table>

Note:  

a) $F = A_p f_p$; $f_p$ (thus $F$) varies with the applied moment $M$.

b) Sign convention: for concrete (compression +; tension -); for steel (tension +; compression -).

c) To obtain the decompression moment, use $f_r = 0$ instead of $7.5 \sqrt{f_c}$. 
Table 3 Summary of solution equations for the cracked section, $M \geq M_{cr}$.

$$\begin{align*}
&\left[\frac{A_{ps}E_{ps}}{3M}(e_{pe} + \varepsilon_{ce})b_w\right]c^3 + \left[b_w - \frac{A_{ps}E_{ps}}{M}(e_{pe} + \varepsilon_{ce})b_w d_p\right]c^2 \\
&+ \left[2(b-b_w)h_f + \frac{2A_{sy}E_s}{E_c} + \frac{2A_{sy}E'_s}{E_c} + 2\frac{A_{ps}E_{ps}}{M}\varepsilon_{cr}\right]c \\
&- \frac{A_{ps}E_{ps}}{M}(e_{pe} + \varepsilon_{ce})c \\
&\left\{2(b-b_w)h_f d_p - (b-b_w)h_f^2 - 2A_{sy}\frac{E_s}{E_c}(d_s - d_p) - 2A_{sy}\frac{E'_s}{E_c}(d'_s - d_p)\right\}c \\
&- \left\{2(b-b_w)h_f^2 + \frac{2}{3}(b-b_w)h_f^2 + 2A_{sy}\frac{E_s}{E_c}(d_s - d_p)d_s \\
&+ 2A_{sy}\frac{E'_s}{E_c}(d'_s - d_p)d'_s - (b-b_w)h_f^2 d_p\right\} = 0
\end{align*}$$

(11)

Solve the above equation for the value of $c$, then compute:

$$f_{cr} = \frac{A_{ps}E_{ps}(e_{pe} + \varepsilon_{ce})c}{\frac{b_w^2}{2}(b-b_w)(c - h_f)^2 - \varepsilon_{cr}A_{ps}\frac{E_{ps}}{E_c}(d_p - c) - A_{sy}\frac{E_s}{E_c}(d_s - c) - A_{sy}\frac{E'_s}{E_c}(d'_s - c)}$$

(12)

$$f_s = \frac{E_s}{E_c}f_{cr}\left(\frac{d_s - c}{c}\right)$$

(13)

$$f_p = f_{pe} + \varepsilon_{cr}E_{ps}\varepsilon_{ce} + \varepsilon_{cr}\frac{E_{ps}}{E_c}f_{cr}\left(\frac{d_p - c}{c}\right)$$

(14)

$$f'_s = \frac{E'_s}{E_c}f_{cr}\left(\frac{c - d'_s}{c}\right)$$

(15)

Note: a) $F = A_{ps}f_p'$; $f_p$ (thus $F$) varies with the applied moment, $M$.

b) For rectangular sections $b = b_w$: if no compressive reinforcement is used, then $A_s' = 0$.

c) Sign convention: concrete (compression +; tension -); steel (tension +; compression -).
Figure 1 — a) Stress and strain diagrams for the uncracked section assuming linear elastic materials. b) Strain change from reference state (assumed uncracked) to cracking moment and definition of bond coefficient $\Omega$. (Diagrams are not to scale).
Figure 2 — a) Stress and strain diagrams for the cracked section (of maximum moment) assuming linear elastic materials. b) Strain change from reference state (assumed uncracked) to maximum service load (cracked) and definition of bond coefficient $\Omega_{cr}$. (Diagrams are not to scale).
Figure 3 — Bond coefficient for: a) uncracked member; b) cracked member in linear elastic range; c) cracked member at ultimate.

\[ \Omega = \frac{2}{\ell \times \Delta M_{\text{max}} \times (\varepsilon_p)_{\text{max}}} \int_0^{\ell/2} \Delta M(x) \times \varepsilon_p(x) \, dx \]

\[ \Omega_{cr} = \frac{\Omega_{cr}}{I_g} \left( 1 - \frac{I_{cr}}{I_g} \right) \]

\[ \Omega_u = \frac{3}{\ell / d_p} \text{ for uniform loading or third point loading} \]
\[ \Omega_u = \frac{1.5}{\ell / d_p} \text{ for single point loading at midspan} \]

Figure 4 — Assumed moment-deflection relationship.
Rational Determination of Friction Losses in Post-Tensioned Construction

by P.R. Gupta

Synopsis: Friction losses contribute about 50% of the total losses in post-tensioned construction. The original loss coefficients that were derived for earlier post-tensioning systems are still being used in the ACI recommendations for unbonded construction. Some of these systems have not been used in the industry for almost 30 years.

This paper presents a review of the current ACI recommendations of friction losses in unbonded construction. The paper also describes the theoretical development of a simple field technique to determine the curvature and wobble coefficients under field conditions. Results from initial testing are compared with the ACI recommendations and industry practice.

Keywords: concrete; curvature; evaluation; force; friction; measurement; prestressing; slab; tendon; tension; unbonded; wobble
ACI Member **Pawan R. Gupta** is a Technical Director at the Post-Tensioning Institute. He received his Ph.D. from the University of Toronto in 1998. He is an member of joint ACI-ASCE Committee 423, Prestressed Concrete, ACI Committee 364 Rehabilitation, ACI Committee 437, Strength Evaluation of Existing Concrete Structures and ACI-440, Fiber Reinforced Polymer Reinforcement.

**INTRODUCTION**

Losses occur during stressing operations in post-tensioned construction. A large portion of the loss in tendon force is associated with friction between the strands and the surrounding sheaths. The design codes give typical values of curvature and wobble coefficient to estimate the losses for some of the commonly used systems. However, depending on the post-tensioning system, the properties of the wrapping and lubricating materials used, project details and workmanship, the friction losses can vary between projects.

An appropriate choice of friction and wobble coefficient is important because this can impact the overall efficiency and influence the cost of the project. The ACI\(^1\) code gives a range of coefficients to determine friction losses. The recommendations for friction and wobble were first introduced in the ACI 318 code in 1963. While there have been significant advances in the post-tensioning technology in the last 30 years, there has been very little change in the ACI recommendations for friction coefficients. Some of the types of strands listed in table R18.6.2 of ACI 318-05 have not been used in the industry for almost 30 years. It is important to review and update these recommendations to reflect the current industry practice.

Clause 18.6.2.2 of ACI 318-05\(^1\) states “Friction loss shall be based on experimentally determined wobble \(K\) and curvature \(\mu\) friction coefficients, and shall be verified during tendon stressing operations.” Traditionally, this has been achieved by assuming a friction and wobble coefficient and then checking the calculated elongation against the measured elongations. Clause 18.20.1 of ACI 318-05 limits the discrepancy of the calculated and field measured elongation to ±7% for post-tensioned members. If the measured elongation is within these limits the assumed loss coefficients are considered to be accurate for the system. If the discrepancy is larger than 7%, the loss coefficients are modified in coordination with the Engineer to give the results that are within the specified range.

The AASHTO “Guide Specification for Design and Construction of Segmental Concrete Bridges”\(^2\) includes the following provision:

> *When specified by the Engineer, the Contractor shall test early in the project, in place, two representative tendons of each size and type shown on the plans, for the purpose of accurately determining the friction loss in strand and/or bar tendon.*
The test procedure shall consist of stressing the tendon at an anchor assembly with load cells at the dead end and jacking end. The test specimen shall be tensioned to 80 percent of ultimate in 10 increments. For each increment, the gauge pressure, elongation and load cell force shall be recorded. The data shall be furnished to the Engineer. The theoretical elongation and post-tensioning forces shown on the post-tensioning shop drawings shall be re-evaluated by the Contractor using the results of the tests and corrected as necessary. Revisions to the theoretical elongations shall be submitted to the Engineer for evaluation and approval. Apparatus and methods used to perform the tests shall be proposed by the Contractor and be subjected to the approval of the Engineer.”

It is noted that the above procedure does not measure the curvature and wobble coefficients separately but measures the total frictional losses for a particular configuration and length of the strand. As will be shown later in the paper, it is important to be able to independently estimate the curvature \( \mu \) and wobble coefficient \( K \) accurately to determine the losses in all situations.

This paper describes a simple field technique to determine the wobble and curvature coefficients in prestressed members. The procedure consists of measuring the force in a tendon at 3 points along the length. The curvature and wobble coefficients are then determined by rearranging the friction loss equation given in the ACI code and solving two simultaneous equations for \( \mu \) and \( K \). Preliminary results with the technique are reported and compared with current industry practice.

LOSSES IN POST-TENSIONED CONSTRUCTION

Tendons in typical P/T construction are draped, see Figure 1. Draping of tenons provides an efficient structural system that makes long span structures economical. The force in a prestressing tendon varies along its length and decreases with time. The losses have traditionally been divided into two types\(^3,4\).

**Instantaneous losses**

Instantaneous losses occur during jacking and anchoring operations. These losses are caused by:

- Friction between strand and sheath
- Elastic shortening of concrete
- Anchorage and seating of wedges
Time Dependent losses

The time dependent losses occur over time and include

- Shrinkage of concrete
- Creep of concrete
- Relaxation of prestressing steel

Losses in unbonded construction can be about 20-25% of the total prestressing force. Frictional losses typically constitute about 50% of these losses and anchorage set, elastic shortening and time dependent losses account for the rest. Zia et al.\textsuperscript{5} have reported a comprehensive study on the estimation of time dependent losses in prestressed concrete systems. This paper will focus on evaluating the frictional losses for unbonded systems.

FRICTION LOSSES

Strands in typical post-tensioned construction are draped. Additional curvatures are also caused due to site conditions. There is a loss in force in the tendon along its length during stressing operations due to friction between the strand and the surrounding sheath. This loss is termed as friction loss. The total friction losses can be divided into curvature and wobble losses.

Curvature Losses

Curvature losses are losses due to the intentional drape of the tendons. The loss in force is caused by the frictional forces that are developed in concrete as the tendons change direction, see Figure 2. The force in the tendon after curvature can be estimated as

\[
(P - d_p) = P e^{-\mu \alpha}
\]

Where \( \mu \) is the coefficient of friction between the tendon sheath and the strand. These losses depend on the material of the sheath, the condition of the interface between the strand and the sheath and the cumulative angle change \( \alpha \). For tendons where the grease has deteriorated frictional losses can be significantly higher.

Wobble Losses

Wobble losses are caused by unintentional change in curvature. In practice it is almost impossible to install perfectly draped tendons, see Figure 3. In addition to curvature, losses are introduced during forming and pouring of the concrete due to the stiffness of the tendons, construction interference, vibration and finishing of concrete. These losses are typically represented as a cumulative angle change per unit length.
The wobble losses are represented as:

\[(P - d_p) = Pe^{-Kx}\]  

(2)

Where \(K\) is the wobble loss coefficient per foot and \(x\) is the length of the tendon.

The total tendon force after frictional loss is conventionally represented as, see Figure 4.

\[P_x = Pe^{-(\mu \alpha + Kx)}\]  

(3)

Where \(P\) is the force in the tendon at the jacking end and \(x\) is the distance where the force is being determined.

**REVIEW OF CURRENT FRICTION RECOMMENDATIONS**

Table 1 shows the ACI recommendations for the curvature and wobble coefficients. It is noted that while there have been significant changes in the technology of fabrication and construction of post-tensioned systems, the friction coefficients recommended by ACI have essentially remained the same for almost 30 years. The mastic-coated unbonded tendons that are still in the ACI recommendations have not been used in the industry for almost 30 years. Similarly 7-wire strands have replaced wire tendons in the industry for almost 20 years. It is important that the design codes reflect current industry practice. The tables need to be updated to incorporate the currently used post-tensioning systems.

Figure 5 shows a typical post-tensioned slab that is 106' in length. The example slab will be used in this paper to review the current code recommendations and industry practice. Equation 3 can be rearranged as

\[\ln \left(\frac{P}{P_x}\right) = -(\mu \alpha + Kx)\]  

(4)

If \(f_{pe}\) is the effective stress in the strand at any point along its length, and \(f_p\) is the stress at the jacking end, Equation 4 can be written as

\[\ln \left(\frac{f_{pe}}{f_p}\right) = -(\mu \alpha + Kx)\]  

(5)

This form of the equation allows us to evaluate the influence of each coefficient \(\mu\) and \(K\) individually.
Figure 5 shows a plot of $\ln \frac{f_{pe}}{f_p}$ at various points along the length of the tendon using $\mu = 0.07$ and $K = 0.001/\text{ft}$. It is noted that about 79% of the total frictional losses are contributed by wobble and only 21% of the losses can be attributed to curvature.

Figure 6 shows the frictional loss profile of an unbonded strand using the range of values given in the ACI code. ACI code gives considerable latitude on the choice of friction coefficients. Depending on the numbers chosen the stress at the dead end could vary between $0.61f_{pu}$ and $0.76f_{pu}$. Most prestressing companies typically use a median value similar to those used in Figure 5. With these values it is interesting to note that wobble losses outweigh the curvature losses by a significant margin for a typical 100 ft long 8 in. thick slab. Although the total elongation corresponds to the measured values, the distribution of the losses is counterintuitive. It is noted that visually the predominant change in geometry of the tendon appears to be from the intended drape of the tendon, see Figure 1. This would suggest the curvature losses to be significantly higher than the wobble losses. This aspect is further discussed later in the paper.

FIELD DETERMINATION OF POST-TENSIONED LOSSES

Because the coefficients for curvature and wobble are nested together in an exponential format in Equation 3, it is difficult to separate the two coefficients. Traditionally the loss coefficients have been determined in two steps\textsuperscript{7}. The wobble loss is first determined by measuring the force at the two ends of a straight tendon using Equation 2. This is then used to determine the curvature coefficient from the curved tendon using Equation 3. Although the procedure gives a reliable estimate of the total losses in the tendon, it does not account for the wobble losses due to the interference of the rebars and other tendons in the field conditions. The problem is caused by physical limitation of measuring the forces in any tendon at the ends only. This results in the determination of only one loss coefficient at a time.

Recently, a field technique was developed to measure the P/T forces at any point along the length of the tendon\textsuperscript{9}. The technique was used to measure the force in a tendon at 3 points. This eliminates the two step process and the curvature and wobble coefficients can be determined for the same strand under field conditions. This paper describes the theoretical development of the technique and some preliminary results.

Field Measurement of Prestressing Force

A simple non-destructive technique to measure the force in existing unbonded tendons was developed by the author\textsuperscript{9}. Figure 7 shows the schematics of the test frame. The technique involves exposing about 2’-0” (610 mm) of the strand along its length. A calibrated lateral force is applied to the strand by a self-supporting frame. A dial gauge measures the lateral deflection of the strand. The force in the tendon is then determined by principles of physics. A detailed description of the technique is given in Reference 9. Figure 8 shows the test-setup in the field. The frame was calibrated in the laboratory and
in the field. Figure 9 shows the results from calibration tests. The test frame is capable of measuring the actual force in the strand with an accuracy of ±5% in the practical range of forces found in typical post-tensioned construction.

Theoretical Development

This section describes the theoretical background for determining the friction and wobble coefficients using the technique described above.

Equation 3 can also be re-arranged as

\[ P = P_x e^{(\mu \alpha + Kx)} \]  

(6)

If the forces \( P_1 \) & \( P_2 \) are measured at 2 points at distance of \( x_1 \) and \( x_2 \) along the length of the tendon, see Figure 10. Equation 6 for each point can be written as

\[ P = P_1 e^{(\mu \alpha_1 + Kx_1)} \quad \text{and} \quad P = P_2 e^{(\mu \alpha_2 + Kx_2)} \]  

(7)

Rearranging the above equation and taking natural logarithm on each side, the equations are reduced to

\[ \ln \frac{P}{P_1} = \mu \alpha_1 + Kx_1 \quad \text{and} \quad \ln \frac{P}{P_2} = \mu \alpha_2 + Kx_2 \]  

(8)

The equations are now de-coupled for \( \mu \) and \( K \). \( P, P_1 \) & \( P_2 \) are measured in the field at points A, B and C, the theoretical values of \( \alpha_1 \) and \( \alpha_2 \) can be calculated from the tendon geometry. The two simulataneous equations can then be solved to determine the coefficients \( \mu \) and \( K \). Figure 10 describes the steps involved in the calculation of \( \mu \) and \( K \).

Field Application

The procedure described above was used in the field to determine the loss coefficients. The project involved replacement of unbonded tendons in a 30 year old office building. Figure 11 shows the dimensions of a typical transverse section of the slab and the tendon profile. The slab was about 106’ in length with two spans of 41’ and cantilevers on each side of about 11’ and 13’ respectively. Paper wrapped tendon system that was common in the 1960’s was used on the project. The overall thickness of the slab was 8” with large drops over the columns. It was decided to measure the friction coefficients for one tendon that was being replaced. Because the ends were not accessible for stressing, an intermediate stressing coupler was used for stressing the tendons. Figure 11 shows the location of the intermediate stressing coupler. The force measurements were taken at three points along the length of the tendon. Figure 11 shows the location and the
measured force at each of the points.

From the existing structural drawings, the total intended curvature change was calculated. Figure 11 shows the calculated angles at \( x_1 = 43' \) and \( x_2 = 78' - 4'' \) from the reference point.

The measured forces \( P, P_1 \) and \( P_2 \) and the calculated curvature change \( \alpha_1 \) and \( \alpha_2 \) were substituted in Equation 8 as:

\[
0.2235\mu + 43K = \ln \frac{17.5}{15.9} \quad \text{and} \quad 0.3129\mu + 78.33K = \ln \frac{17.5}{15.2}
\]

The two simultaneous equations were solved to give

\[
\mu = 0.358 \quad \text{and} \quad K = 0.00037
\]

Figure 11 summarizes the calculation procedure for the loss coefficients.

**DISCUSSION OF TEST RESULTS**

It is noted that the field measured values of curvature coefficient are larger than the ACI recommendations while the wobble coefficient was smaller. Figure 12 shows the distribution of losses between wobble and curvature along the length of the tendon. It is noted that only 18% of the losses are determined to be contributed by wobble while 82% of the frictional losses were contributed by curvature. This is consistent with the observations made earlier. The calculated and measured elongations agreed using the measured friction coefficients.

Figure 13 shows the force profile of the tendon using the calculated friction coefficients, the commonly used coefficient of \( \mu = 0.07/\text{rad} \) and \( K = 0.0010/\text{ft} \) and the maximum ACI recommended coefficients of \( \mu = 0.15/\text{rad} \) and \( K = 0.0020/\text{ft} \) for the example tendon. It is noted that each of the above procedures would satisfy the \( \pm 7\% \) ACI requirement.

Cooley\textsuperscript{7,8} conducted extensive investigation on the friction losses in different post-tensioned systems. The ACI recommendations for the friction coefficients were based on his original work. In his original report Cooley, reported that the friction coefficient of bare strand on an unfolded piece of sheathing material was measured to be higher than curvature coefficient as deduced by the two tendon method described earlier. Although in theory the conventional technique for determining curvature and wobble coefficients is sound, it is believed that experimental errors may have contributed to a higher measurement of wobble coefficient, which resulted in a lower curvature coefficient.

A theoretical study was conducted to evaluate the influence of loss coefficients on a typical 8” thick 100’ long slab using various coefficients used in the industry. Table 2
summarizes the results of the study. It is noted that regardless of the values $\mu$ and $K$ used, the force in the tendon at the dead end ranges between $0.67f_{pu}$ and $0.7f_{pu}$. However, the distribution of the losses measured in the test is significantly different to that used in the industry. It is also noted that the elongations calculated from all of the coefficients used in the industry would easily satisfy ACI limit of 7%.

CONCLUSIONS

- Curvature $\mu$ and wobble $K$ friction coefficients can be experimentally determined using the technique suggested.
- More work is required to understand and quantify all of the factors that affect the curvature $\mu$ and wobble $K$ coefficients.
- Current ACI recommendations for evaluating frictional losses are outdated. They recommendations need to be updated to reflect current industry practice.

ACKNOWLEDGMENTS

The work was done when the author was employed with Halsall Associates Limited, Toronto, Canada. The author would like to acknowledge their support in the development and testing of the technique presented in this paper.

REFERENCES

1. ACI Committee 318, “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)”, American Concrete Institute, Farmington Hills, Mich., 430 pp.


Table 1 — ACI recommendations for curvature and wobble coefficients

<table>
<thead>
<tr>
<th></th>
<th>Wobble Coefficient, K/ft</th>
<th>Curvature Coefficient, ( \mu_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grounded tendons</td>
<td></td>
<td></td>
</tr>
<tr>
<td>In metal sheathing</td>
<td>Wire Tendons</td>
<td>0.0010-0.0015</td>
</tr>
<tr>
<td></td>
<td>High-strength bars</td>
<td>0.0001-0.0006</td>
</tr>
<tr>
<td></td>
<td>7-wire strand</td>
<td>0.0005-0.0020</td>
</tr>
<tr>
<td>Unbonded</td>
<td>Wire Tendons</td>
<td>0.0010-0.0020</td>
</tr>
<tr>
<td>Mastic coated</td>
<td>7-wire strand</td>
<td>0.0010-0.0020</td>
</tr>
<tr>
<td>Pre-greased</td>
<td>Wire Tendons</td>
<td>0.0003-0.0020</td>
</tr>
<tr>
<td></td>
<td>7-wire strand</td>
<td>0.0003-0.0020</td>
</tr>
</tbody>
</table>

Adapted from ACI-318-2005
Table 2 — Typical range of frictional losses used in the industry

<table>
<thead>
<tr>
<th></th>
<th>ACI Min(^1)</th>
<th>ACI Max(^1)</th>
<th>PTI(^2)</th>
<th>Can BBR(^3)</th>
<th>VSL(^4)</th>
<th>PTDATA(^5)</th>
<th>DSI</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\mu)</td>
<td>0.05</td>
<td>0.15</td>
<td>0.07</td>
<td>0.05</td>
<td>0.05</td>
<td>0.07</td>
<td>0.07</td>
<td>0.36</td>
</tr>
<tr>
<td>K/ft</td>
<td>0.0003</td>
<td>0.0020</td>
<td>0.0010</td>
<td>0.0015</td>
<td>0.0014</td>
<td>0.0010</td>
<td>0.0012</td>
<td>0.0004</td>
</tr>
<tr>
<td>(h=6)</td>
<td>(L=106)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(\mu)</td>
<td>38%</td>
<td>22%</td>
<td>21%</td>
<td>11%</td>
<td>12%</td>
<td>21%</td>
<td>18%</td>
<td>82%</td>
</tr>
<tr>
<td>K</td>
<td>62%</td>
<td>78%</td>
<td>79%</td>
<td>88%</td>
<td>88%</td>
<td>79%</td>
<td>82%</td>
<td>18%</td>
</tr>
<tr>
<td>(P_e) (kips)</td>
<td>31.4</td>
<td>25.2</td>
<td>28.9</td>
<td>27.6</td>
<td>27.9</td>
<td>28.9</td>
<td>28.3</td>
<td>27.8</td>
</tr>
<tr>
<td>%(\mu_p)</td>
<td>76%</td>
<td>61%</td>
<td>70%</td>
<td>67%</td>
<td>68%</td>
<td>70%</td>
<td>69%</td>
<td>67%</td>
</tr>
</tbody>
</table>

\(^1\) Provided by Canadian BBR (Private Communication)
\(^2\) Provided by VSL (Private Communication)
\(^3\) PTDATA Manual

Figure 1 — Draped tendons in post-tensioned construction

Figure 2 — Curvature frictional loss\(^3\)
Figure 3—Wobble frictional loss

Figure 4—Total friction loss calculation

Figure 5—Distribution of frictional losses along the length of a tendon
Figure 6— Range of frictional losses

Figure 7— In-situ tension test frame
Figure 8— In-situ tension test set-up in field

Figure 9— Calibration of in-situ tension tester
Figure 10—Procedure for calculation of curvature and wobble coefficients

\[ P = P_1 e^{(\mu \alpha_1 + Kx_1)} \]
\[ \log \left( \frac{P}{P_1} \right) = \mu \alpha_1 + Kx_1 \]

Field Measure \( P, P_1 \& P_2 \)

Solve for \( \mu \) & K

Figure 11—Field determination of curvature and wobble coefficients

<table>
<thead>
<tr>
<th>Location</th>
<th>( \Delta ) (in.)</th>
<th>Force (kips)</th>
<th>Location**</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.3581</td>
<td>15.2</td>
<td>83'-7&quot;</td>
<td>Stressing jack maintained at 3500 psi</td>
</tr>
<tr>
<td>B</td>
<td>0.3704</td>
<td>15.9</td>
<td>48'-3&quot;</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>0.3368</td>
<td>17.5</td>
<td>5'-3&quot;</td>
<td></td>
</tr>
</tbody>
</table>

*Average of three readings
**Distance of measuring point from stressing jack.
Figure 12—Measured friction and wobble losses

Figure 13—Range of friction losses
Effects of Friction and Slip-Back on Stresses in Post-Tensioning Tendons

by J.F. Stanton

Synopsis: In post-tensioned systems, friction during stressing and slip-back due to setting the wedge anchors cause loss of prestress. If the tendon is long or contains sharp curvatures, these losses can be significant. This paper summarizes methods for calculating the losses and provides an evaluation of the numerical coefficients suggested by ACI 318-05 for friction. Equations are provided where closed form methods are possible, and numerical methods are outlined for other cases.

Keywords: anchorage set; friction; post-tension; prestress loss
INTRODUCTION

When a tendon is post-tensioned, the stress along it varies because of friction along the length and slip-back at the anchorage. These behaviors represent instantaneous losses, which must be added to the subsequent changes in tendon stress due to creep, shrinkage, etc. The purpose of this paper is to summarize the effects of friction and slip-back, to present methods of computing them and to offer some suggestions for stressing strategies that take account of their effects. The information presented here presupposes the availability of reliable values of the friction properties of the system.

DEVELOPMENT OF EQUILIBRIUM EQUATIONS

Friction in a tendon is usually characterized as having one component due to curvature and another due to “wobble.” The first is obtained as the friction caused by the normal force induced when a cable passes round a curved path, as illustrated in Figure 1. Equilibrium of a segment of length $dz$, in the directions perpendicular and parallel to the tendon axis respectively, gives

\[ N = T \frac{d\theta}{dz} \]  \hspace{1cm} (1a)
\[ dT = -F = -\mu |N| = -\mu T \frac{d\theta}{dz} \]  \hspace{1cm} (1b)

The absolute value sign arises because friction reduces the tendon force regardless of the sense of the change in angle. Wobble friction describes the loss in force per unit length of a nominally straight tendon. It may be caused by either adhesion of the tendon to the sides of the duct or, more likely, to accidental curves in the tendon path. It is expressed as $K$, a loss rate per unit length. When combined with Equations 1a and b, the result is

\[ \frac{dT}{dz} = -\left( K + \mu \frac{d\theta}{dz} \right) T = -\lambda T \]  \hspace{1cm} (2)

where $\lambda = K + \mu \left| \frac{d\theta}{dz} \right|$  \hspace{1cm} (3)

In Equation 2, the term $d\theta/dz$ is the change in slope per unit length, which can be interpreted as the curvature of the tendon profile.
EXACT AND APPROXIMATE SOLUTIONS FOR FRICTIONAL EFFECTS

If the tendon path consists of a single parabolic curvature, as might be the case for a draped tendon in a simply supported beam, the exact solution of Equation 2 is given by

\[ T(z) = T_0 \exp\{ -\lambda z \} \]  

(4)

where \( T_0 \) is the force at the jacking end.

If \( \lambda z << 1.0 \), Equation 4 may be expanded in a Taylor Series and truncated to give the linear approximation

\[ T(z) \approx T_0 \left( 1 - \lambda z \right) \]  

(5)

For a tendon profile that can be represented by a general, \( n \)th order, polynomial, \( p_n(z) \), the curvature, \( \phi(z) \), is given by \( p_n''(z) \), and the distribution of tendon force is

\[ T(z) = T_0 \exp\left\{ - \int_0^z \phi(s) ds \right\} = T_0 \exp\left\{ - \int_0^z p_n''(s) ds \right\} \]  

(6)

For example, Figure 2 shows the end span of a continuous beam, in which the tendon profile is a cubic. It passes through the cg at the end, is horizontal over the first interior support, and maintains 2” cover to the cg at both the high and the low points. The profile is given by

\[ y(\zeta) = 104.1(\zeta - 2.173\zeta^2 + 1.115\zeta^3) \]  

(7)

where \( \zeta = z/L \)

The curvature is

\[ y''(\zeta) = \frac{104.1}{L^2}(-4.346 + 6.690\zeta) \]  

(8)

which changes sign at \( \zeta = 0.6494 \). The integral of Equation 6 must be evaluated separately for \( 0 < \zeta < 0.6494 \) and \( 0.6494 < \zeta < 1.0 \) in order to include the absolute value. The slopes of the profile at \( \zeta = (0, 0.6494, 1.0) \) are found to be \( (0.2890, -0.1189, 0.0) \), from which the total absolute change in angle over the span is 0.5267 radians, and

\[ \frac{T(L)}{T(0)} = \exp(-0.0001*360 + 0.2*0.5267) = 0.8682 \]  

(9)

\( T \) is plotted as a function of \( \zeta \) in Figure 3. The equivalent load, \( w_{eq} \), can be obtained from the tendon force and profile. It is a concept that is widely used in designing continuous systems, because it avoids the need for computing secondary moments when considering service load behavior. It is given by
\[ w_{eq} = \frac{d(e_p T')}{dz} = T e_p ' + T' e_p ' \]  

(10)

where the prime indicates differentiation with respect to \( z \). \( w_{eq} \) is plotted against \( z \) in Figure 4. In the absence of friction losses, it would be exactly linear. The effects of the losses are manifested in both terms in Equation 10, because \( T \) changes with \( z \). In this example, as in most others, the second term is quite small compared with the first (< 9% in this case), and is often neglected.

**FRICTION VALUES**

The Commentary to ACI 318-05 gives values for wobble and curvature friction, which have been unchanged since the 1972 code. Their consistency may be examined based on the assumption that the friction attributed to wobble really arises from accidental curvature. The implied curvature is then

\[ \phi(z) = \frac{d\theta}{dz} = \frac{K}{\mu} \]  

(11)

Values of \( K, \mu \) and the implied \( \phi \) are given in Table 1 for various tendon types. The lack of a consistent pattern in the implied curvatures suggests that uncertainty exists in the suggested friction values.

The tendon types in most common use today are 7-wire strands either in metal ducts that are grouted after stressing and 7-wire strands in greased sheaths. The Commentary advises that for strands in rigid or semi-rigid ducts, the wobble coefficient may be taken as zero. This may be justified on the basis that the duct is sufficiently rigid that the accidental curvatures will be very small. Such ducts are often used in elements such as box girders, wherein the ducts are essentially constrained to exist in the plane of the web. This in itself permits lower accidental curvature values than are likely to occur in a greased mono-strand placed in a slab. There, the strands often curve in both the horizontal plane (to avoid obstructions such as openings) and the vertical plane (to provide upward load to balance gravity loads). The Commentary is silent on the question of whether the wobble coefficient should be used to represent both known horizontal curvatures and accidental curvatures, or only the latter. For thin slabs with numerous openings, the horizontal curvatures may be larger than the vertical ones, in which case it is rational to evaluate both curvatures explicitly, and to use the wobble coefficient to address only the truly accidental curvatures.

**EFFECTS OF SLIP-BACK**

Slip-back occurs when the wedges of a strand anchorage are set. In most multi-strand rams, the slip-back distance is a compromise between two opposing goals, and can be adjusted. If it is set too small, the strand drags on the wedge during stressing and can be
damaged. If it is set too large, excessive strand stress is lost when the wedge is set. Values between about 3/16” and 3/8” are common.

Slip-back at the wedge causes the strand to slip back into the duct, and the stress drops below the jacking stress. In the absence of friction, the slip would extend along the whole strand, and the stress would drop by the same amount everywhere. However, friction prevents this from occurring, so the slip may penetrate over only part of the span. Two cases occur. In so-called “short beams”, the slip extends over the whole length, and the stress drops at the dead end anchorage. In “long beams”, the slip penetrates over only part of the span, and the stress at the dead end does not change.

**Long beam**

**Single segment profile** - Consider the simplest case of a tendon with parabolic profile. Directly after jacking, and prior to any slip back, the tendon force varies with z as

\[ T(z) = T_j \exp\{-\lambda z\} \]  

When slip occurs, the friction reverses and the force distribution in the slipped region is

\[ T(z) = T_i \exp\{\lambda z\} \]  

where \( T_i = \) the initial tension at the live end directly after setting the wedges.

If the slip penetrates to a point \( z = b \), the stress force just to the right and left of \( z = b \) must be the same. Therefore

\[ T_j \exp\{-\lambda z\} = T(b) = T_i \exp\{\lambda z\} \]

so

\[ \frac{T_j}{T_i} = \exp\{-2\lambda b\} \]  

The drop in force at any point \( z < b \) is given by

\[ \Delta T = T_j \exp\{-\lambda z\} - T_i \exp\{\lambda z\} = T_j \left(\exp\{-\lambda z\} - \exp\{-2\lambda b + \lambda z\}\right) \]

Thus the slip back distance, \( u_{slip} \), can be expressed as

\[ u_{slip} = \int_0^b \frac{\Delta T}{A_p E_p \lambda} \, dz = \frac{f_j}{E_p \lambda} \left(1 - \exp\{-\lambda b\}\right)^2 \]

where \( f_j = \) the jacking stress in the tendon.

Since \( u_{slip} \) is known and the distance \( b \) is not, Equation 17 may be inverted to give
The drop in stress at the jacking end, from $T_j$ to $T_i$, is of primary interest. Equations 16, 18 and 19 can be combined to give

$$\frac{T_i}{T_j} = (1 - \eta)^2$$

(20)

and

$$\frac{T(b)}{T_j(0)} = \frac{T_i(0)}{T(b)} = (1 - \eta)$$

(21)

At $z = b$ the stress is the same at jacking (before slip-back) and under initial conditions (after slip-back), so no subscript is used.

Because $\eta << 1.0$, Equation 18 may be linearized by expanding the $\ln$ term and truncating the series, to give

$$b \approx \frac{\eta}{\lambda} = \frac{u_{slip}E_p}{\lambda f_j}$$

(22)

However, this approximation is not much simpler than the exact value (Equation 18), so, although several authors present it, it is of little real interest.

Example

Let $\lambda = 0.001$ in$^{-1}$, $E_p = 28,500$ ksi, $f_j = 200$ ksi and $u_{slip} = 1/8$”. Then Equations 18 and 19 predict slip-back penetration distances of 143.25” and 133.46” respectively. Figure 5 illustrates the distribution of stress before and after slip-back, using both the exact and approximate procedures. The slip back distance, $u_{slip}$, can be interpreted as $(1/E_p)$ times the triangular area.

Multi-segment profile - In a continuous beam or slab, the tendon profile is likely to be made up from a number of individual segments. Use of parabolic segments offers the advantage that the equivalent load is very nearly uniform, thereby counteracting closely what is likely to be a uniformly distributed gravity load. Figure 6 shows the end spans of a multi-span slab, which is supported on beams. The tendon profile consists of upward curving parabolas in the spans, and down-curving parabolas over the beams. Figure 7 shows the corresponding stress distribution in the tendon before and after slip-back. (The
data used was: clear span = 180”, beam width = 20”, \( \lambda = 0.00015 \text{ in}^{-1} \) in the spans, \( \lambda = 0.0006 \text{ in}^{-1} \) over the beams, \( u_{\text{slip}} = 0.375 \text{ in.} \), \( f_j = 200 \text{ ksi} \), \( E_p = 28,500 \text{ ksi} \). The slip-back penetrates for a total distance of 570 in, or nearly to the end of the third clear span.

The slip-back distance is found using the same principles as for a single segment profile, but the numerical computations have to be modified to deal with the local changes in \( \lambda \). The area of the approximately triangular region between the jacking and initial stress curves is once again equal to the product \( E_p * u_{\text{slip}} \). However, the value of \( b \) cannot easily be found in closed form, and is most easily determined numerically. This is done by computing the areas between \( f_j \) and \( f_i \) within each segment and adding the results. If the jacking stresses at the start and end of segment \( n \) are \( f_{j,n-1} \) and \( f_{j,n} \), and the initial stresses (after slip-back) are \( f_{i,n-1} \) and \( f_{i,n} \), then the area within segment \( n \) is

\[
A_n = \frac{f_{j,n-1}}{\lambda_n} \left[ 1 - e^{-\lambda_n L_n} \right] + \frac{f_{i,n-1}}{\lambda_n} \left[ 1 - e^{\lambda_n L_n} \right] \\
= \frac{1}{\lambda_n} \left( f_{j,n-1} - f_{j,n} \right) - \left( f_{i,n} - f_{i,n-1} \right)
\]  

(23)

Equation 23 gives the exact value. An approximate value may be obtained by treating the region as a trapezoid, which leads to

\[
A_n \approx \frac{L_n}{2} \left( f_{j,n-1} - f_{i,n-1} \right) + \left( f_{j,n} - f_{i,n} \right)
\]  

(24)

The jacking and initial stresses at each point (at the start and end of each segment) are obtained by assuming a specific value of \( b \) and using, for each segment, the exact relationships

\[
\frac{f_{j,n}}{f_{j,n-1}} = \exp \left( -\lambda_n L_n \right)
\]  

(25a)

\[
\frac{f_{i,n-1}}{f_{i,n}} = \exp \left( -\lambda_n L_n \right)
\]  

(25b)

In the segment in which the slip-back ends, the distance from the left end of the segment to the end of the slip-back region is substituted for \( L_n \). The total area is then computed by adding the areas \( A_n \), and then the \( u_{\text{slip}} \) value calculated from Equation 18 is compared with the true (known) value. The value of \( b \) is then adjusted until the target and calculated values of \( u_{\text{slip}} \) agree within an acceptable tolerance. This can easily be done on a spreadsheet or other program.

In some cases the segments may consist of profiles that are not parabolic. Then, solving for the exact area of the segment becomes more difficult, and the approximation of Equation 24 may be used. The error is likely to be small unless the curvatures or friction values are unusually high. If necessary, the (exact) stresses may be computed at
intermediate points using Equation 6 and the appropriate polynomial expression for the tendon profile, and the area of the segment may be computed to any desired accuracy using the trapezoidal rule. In practice, the friction coefficients are unlikely to be known to an accuracy that would justify this effort.

**Short beam**

In a short beam, the slip-back may extend to the far end. Then some variable other than $b$, the slip-back penetration distance, is needed to characterize the stress distribution. The stress drop at the dead end anchorage, $\Delta f_{\text{dead}}$, may be used. Equation 23 or 24, with $n = 1$, gives the area of the region between the jacking and in initial stress curves. The stress drop at the dead end anchorage (location 1) is

$$\Delta f_{\text{dead}} = (f_{j,1} - f_{i,1}) \quad (26)$$

The initial stress at the live end, $f_{i,0}$, is obtained from $f_{i,1}$, using Equation 4 or 6. $\Delta f_{\text{dead}}$ is then adjusted until it is equal to $E_p \ast u_{\text{slip}}$. This procedure gives the stress distribution at all points along the tendon.

**EVALUATION OF STRESSING STRATEGIES**

In members that suffer severe friction or slip-back losses, the member is sometimes stressed from both ends in an attempt to overcome the losses. The effectiveness of doing so is examined here.

Consider the simplest case of a parabolic tendon in a beam, illustrated in Figure 8. In the figure, stresses after jacking but before slip-back are shown as open symbols, and after slip-back (at “initial conditions”) they are shown as solid symbols. The final profile, after stressing from both ends and accounting for slip-back at both ends, is shown as a cross. For the case shown, $b = 0.2L$, $\lambda L = 0.1$ and $f_{j,0} = 200$ ksi.

After stressing from the first end (the left, as shown here), the lowest stress occurs at the right end of the beam and is

$$f_{\text{min,1}} = f_{j,0} \exp(-\lambda L) \quad (27)$$

After stressing from the right end, the lowest stress still occurs at the right end, but now has the value

$$f_{\text{min,2}} = f_{j,0} \exp(-2\lambda b) \quad (28)$$

The ratio of the stresses from Equations 27 and 28 indicates the benefit gained by stressing from both ends, and is

$$f_{\text{min,ratio}} = \frac{f_{\text{min,2}}}{f_{\text{min,1}}} = \frac{f_{j,0} \exp(-2\lambda b)}{f_{j,0} \exp(-\lambda L)} = \exp(2\lambda(L / 2 - b)) \quad (29)$$
Equation 29 shows that if $b < L/2$, stressing from both ends raises the minimum prestress level and provides a clear benefit. It also raises the average prestress level. As the slip-back distance, $b$, approaches $L/2$, the ratio in Equation 29 drops to 1.0, so the benefit diminishes, and using more tendons may be a more cost-effective strategy than stressing from both ends. The choice will depend on the costs of materials and installation relative to the costs of stressing. (The number of tendons must be selected at design time, so an estimate of the friction characteristics of the system must be made then in order to determine the best strategy).

If $b > L/2$, stressing from both ends offers no benefit, because the final stress distribution is just the reverse of what it was after the first end was stressed. As shown by Equation 20, the slip-back penetration distance, $b$, depends on the magnitude of the slip, $u_{slip}$.

If the tendon profile is non-symmetric and stressing is to be conducted from one end only, it should be done from the end where the curvature is lowest. Doing so does not change the minimum prestress level (which occurs at the dead end), but it raises the average prestress.

**EFFECTIONS ON FRICTION OF CHANGE IN CURVATURE**

The variation in stress along a tendon occurs because of friction, and that friction is primarily associated with the normal force that arises from curvature of the tendon. It is thus reasonable to enquire whether the stress along the tendon may become more uniform as the curvature of the beam changes under loading. In the extreme, a beam that is bent through a curvature that is equal and opposite to that of the original tendon profile would render the profile straight, which would in turn eliminate all “curvature” friction. In the absence of “wobble” friction, the stress in the tendon would then become completely uniform. Of course, such a redistribution of stress can occur only in unbonded tendon systems.

The extent of the change to be expected in practice can be investigated analytically. Consider a tendon with a circular profile, such as might be used for stressing a tank. The length is $L$, the coefficient of friction is $\mu$, the radius is $R$, and the curvature of the path is therefore $1/R$. Wobble friction is ignored here, and in the interests of simplicity an idealized anchor with no slip-back is used. After stressing, the stress along the tendon is given by

$$f(z) = f_{j,0} \exp(-\mu\phi z)$$  \hspace{1cm} (30)

The elastic elongation of the tendon is

$$\delta L = \int_0^L \varepsilon(z)dz = \frac{f_{j,0}}{E_p/\mu \phi} \left\{1 - \exp(-\alpha)\right\}$$  \hspace{1cm} (31)

where $\alpha = \mu \phi L$

If the beam is bent through a curvature that is constant along its length, the change in $\phi$ may be expected to alter the stress distribution. Differentiating Equation 31 gives:
If the member does not change length when the curvature changes, Equation 32 must be equal to zero, which leads to

\[
\frac{df_{j,0}}{f_{j,0}} = \frac{1}{E_p \mu} \left\{ \left[ \frac{1}{\phi} - \left( 1 - e^{-\alpha} \right) \right] df_{j,0} + \left[ \frac{1}{\phi^2} - \frac{1}{\phi} \left( 1 + \alpha e^{-\alpha} \right) \right] f_{j,0} \phi \right\}
\]

Equation 33 relates the change in stress at the jacking end to the change in curvature for a member with a circular tendon profile. Little error is induced by applying it to a parabolic profile in a straight member. Consider, for example, a 60 ft. long member that has a parabolic tendon profile with a central sag of 14 in. and \( \mu = 0.2 \). Then \( \phi = 0.000216 \) rad/inch, \( \alpha = 0.0311 \) and, using Equation 33,

\[
\frac{df_{j,0}}{f_{j,0}} = 0.0155 \frac{d\phi}{\phi}
\]

For this member, the largest change in load-induced curvature that is consistent with ACI allowable stresses is about \( 3.5 \times 10^{-6} \) rad/inch. Therefore, for that applied curvature and an assumed \( f_{j,0} = 200 \) ksi, Equation 34 leads to

\[
\frac{df_{j,0}}{f_{j,0}} = 0.0155 \frac{d\phi}{\phi} f_{j,0} = 0.0155 \frac{0.000035}{0.000216} 200 = 0.5 \text{ksi}
\]

Thus the loading causes a small amount of slip to take place, and the peak stress (at the jacking end) drops by about 0.5 ksi, while the lowest stress (at the dead end) increases by about 0.5 ksi. The stress at mid-length remains essentially constant. If the load is released and the member is bent back in the other direction, no slip occurs in the opposite direction, and the stresses along the tendon remain unchanged. Further cycling of the displacements has no effect.

To evaluate the significance of this stress, its change may be compared with the variation of stress along the member, given by Equation 4. \( \lambda L = \alpha = 0.0311 \), so the stress during jacking at the dead end is

\[
f_{j,1} = 200 \exp(-0.0311) = 194 \text{ksi}
\]

The stress change of 0.5 ksi caused by the slip that occurs during service is less than 10% of the 6 ksi variation of stress along the tendon caused by the friction during jacking. The stress change due to in-service slip is thus too small to be of practical significance, given the uncertainties inherent in the coefficients of friction.
The analysis therefore shows that the non-uniform distribution of stress along the tendon that is introduced by the effects of friction and slip-back must be expected to remain largely in place even after cyclic loading of the member. No formal experimental evidence is known of to substantiate this finding. However, anecdotal evidence from experiments in which inelastic cyclic loading was applied to an unbonded frame sub-assemblage (Day et al., 2000) supports it.

SUMMARY AND CONCLUSIONS

The effects on tendon stress of friction along the tendon and slip-back at the anchorage were examined. Closed-form equations were provided where possible, and approximate numerical methods were suggested for other cases, for computing the effects of the losses. Some of the equations represent standard methods; the purpose of this paper is to summarize them all in one place.

It was shown analytically that:

1. Stressing a tendon from both ends provides benefits only if the slip-back penetration distance is less than half the tendon length.
2. Whenever possible, stressing should be conducted from the end where the curvature of the tendon profile is the lowest.
3. The non-uniform distribution of stress caused by friction and slip-back during stressing will be affected only slightly by the effects of subsequent lateral loading of the member. Complete re-distribution to a uniform stress along the tendon will not occur.

REFERENCES

ACI Committee 318, 2005, “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05),” American Concrete Institute, Farmington Hills, Mich., 430 pp.


NOTATION LIST

\[ A_n = \text{for segment } n \text{ of the tendon profile, area between the curves for jacking stress and initial stress} \]
\[ A_p = \text{cross-sectional area of tendon} \]
\[ b = \text{slip-back penetration distance} \]
\[ E_p = \text{Young’s modulus for tendon} \]
Stanton

\( e_p \) = eccentricity of tendon with respect to cg.

\( F \) = friction force.

\( f_i \) = initial stress in tendon (after slip-back).

\( f_j \) = jacking stress in tendon (before slip-back).

\( f_{\text{min},1} \) = minimum stress in tendon after stressing from the first end.

\( f_{\text{min},2} \) = minimum stress in tendon after stressing from the second end.

\( K \) = wobble friction coefficient per unit length.

\( L \) = span length.

\( L_n \) = length of segment \( n \) of tendon profile.

\( N \) = transverse force.

\( n \) = segment number.

\( p_n(z) \) = \( n \)th order polynomial in \( z \).

\( s \) = dummy length variable.

\( T \) = tension force in tendon.

\( T_0 \) = tension force at jacking end of tendon.

\( T_j \) = tension force in tendon under initial conditions (after slip-back).

\( T_j \) = tension force in tendon at jacking (before slip-back).

\( u_{\text{slip}} \) = slip back of wedge anchor.

\( w_{\text{eq}} \) = equivalent transverse load.

\( y \) = vertical ordinate of tendon profile.

\( z \) = longitudinal co-ordinate.

\( \Delta f_{\text{dead}} \) = change in stress at dead end of tendon.

\( \alpha \) = total angle change along tendon segment.

\( \delta L \) = tendon elongation.

\( \phi(z) \) = curvature of tendon profile.

\( \eta = \sqrt{\frac{u_{\text{slip}} E_p \lambda}{f_j}} \)

\( \lambda = K + \mu \left| \frac{d\theta}{dz} \right| \)

\( \mu \) = coefficient of friction between tendon and duct.

\( \theta \) = slope of tendon profile.

\( \zeta = \frac{z}{L} \)

Table 1. ACI friction coefficients and implied curvatures.

<table>
<thead>
<tr>
<th>Sheathing Type</th>
<th>Tendon Type</th>
<th>Wobble coeff, ( K ) (ft.²)</th>
<th>Curvature coeff, ( \mu )</th>
<th>Implied curvature (rad/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gritted tendons in metal sheathing</td>
<td>Wire tendons</td>
<td>0.0010-0.0015</td>
<td>0.15-0.25</td>
<td>0.0067-0.0060</td>
</tr>
<tr>
<td></td>
<td>High-strength bars</td>
<td>0.0001-0.0006</td>
<td>0.08-0.30</td>
<td>0.00125-0.00020</td>
</tr>
<tr>
<td></td>
<td>7-wire strand</td>
<td>0.0005-0.0020</td>
<td>0.15-0.25</td>
<td>0.0033-0.0080</td>
</tr>
<tr>
<td>Unbonded tendons</td>
<td>Mastic coated</td>
<td>Wire tendons 0.0010-0.0020</td>
<td>0.05-0.15</td>
<td>0.020-0.013</td>
</tr>
<tr>
<td></td>
<td>Pre-greased</td>
<td>7-wire strand 0.0010-0.0020</td>
<td>0.05-0.15</td>
<td>0.020-0.013</td>
</tr>
<tr>
<td></td>
<td>Wire tendons</td>
<td>0.0003-0.0020</td>
<td>0.05-0.15</td>
<td>0.0069-0.0133</td>
</tr>
<tr>
<td></td>
<td>7-wire strand</td>
<td>0.0003-0.0020</td>
<td>0.05-0.15</td>
<td>0.0069-0.0133</td>
</tr>
</tbody>
</table>
Figure 1. Equilibrium of a short tendon segment

Figure 2. Tendon profile for example.

Figure 3. Tendon force distribution along span.
Figure 4. Variation of equivalent load along span (− = upwards).

Figure 5. Loss of prestress due to slip-back at anchor.

Figure 6. End spans of a multi-span slab.
Figure 7. Effects of slip-back at anchor in multi-segment profile.

Figure 8. Effects of stressing from both ends.
A Comparison of Methods for Experimentally Determining Prestress Losses in Pretensioned Prestressed Concrete Girders

by E. Baran, C.K. Shield, and C.E. French

Synopsis: This paper presents a description and comparison of several experimental techniques used to determine the effective prestressing force in pretensioned prestressed concrete girders. The effective prestressing force was determined by three methods: (1) using vibrating wire strain gages that were embedded in the girders during fabrication; (2) load testing the girders to determine flexural cracking and crack re-opening loads and then back calculating the losses; and (3) exposing a length of strand, attaching resistance strain gauges on the strands, and flame-cutting the instrumented strands.

Several instruments were used to determine the flexural crack initiation and crack re-opening loads. These included crack detection gages, concrete surface strain gauges, and LVDTs, as well as visual observations. Use of data from the strain gauges placed at the bottom surface of the girders was determined to be the most effective way of detecting flexural crack initiation and re-opening. Cracking loads determined from visual observation were significantly larger than those determined from the strain gauge data.

The back-calculated prestress losses determined from the measured flexural crack initiation and re-opening loads of the girders were significantly larger than those determined from other experimental methods and those predicted by the PCI Committee and AASHTO LRFD methods. Losses determined by the other experimental methods and the predictions correlated more closely with those back-calculated using visually-observed cracking loads.

These results indicate that prediction of losses based on the measured flexural cracking and crack re-opening loads using the basic theory of mechanics results in an overestimation of prestress losses. Consequently, girders may undergo flexural cracking and crack re-opening at lower loads than predicted using the basic theory of mechanics.

Keywords: flexural cracking; prestress losses; prestressed concrete girders; testing
INTRODUCTION

Durability is an inherent requirement of prestressed concrete bridge girders as these members are designed to remain uncracked during service. Loss of prestress occurs, however, both during the production and over the service life of prestressed concrete girders, which affects the performance of the girders. Underprediction of these losses during the design phase may cause adverse effects, such as inaccurate prediction of girder camber, section stresses, and shear resistance of a section. In extreme cases, underprediction of prestress losses may even cause the girders to crack under service loads. Designing girders with overpredicted losses, on the other hand, may result in inefficient use of prestressed concrete.

Prestress losses in a girder can be determined either numerically or experimentally. Numerical determination of prestress losses includes prediction of total losses that occur from the time that the strands are tensioned in the prestressing bed. The total prestressing loss at any time includes losses due to anchorage seating in the prestressing bed, elastic shortening of the strands during strand release, relaxation of the steel, and creep and shrinkage of the concrete over time. Even though there are several methods available to estimate the value of each loss component, due to the interdependent nature of the components, the losses predicted by these methods may not give the correct value. In addition to this, to obtain the best results from these methods, the environmental and loading conditions that the girder is subjected to over time must be known.

Experimental methods commonly used by researchers to determine prestress losses include: 1) direct monitoring of the concrete strain inside the girder over time at the level of the center of gravity of the prestressing strands using embedded vibrating wire strain gages, and using this information to compute the prestress losses; 2) load testing the girder to determine the flexural crack initiation and/or crack re-opening loads, and then back-calculating the effective prestress and prestress losses using the basic theory of mechanics; and 3) exposing a portion of one or more of the prestressing strands, placing strain gages on the strands, subsequently cutting the strands, and computing the effective prestress and prestress losses using the measured strand strain change. Each of these methods has some drawbacks. Method 1 can only be used with girders instrumented during fabrication. Method 2 is highly dependent on the technique used to determine crack initiation and re-opening loads of the girder. Method 3, which is the most direct way of determining the effective prestress existing in girder, is a destructive method.
While Method 1 can be used to determine changes in prestresses losses over time, Methods 2 and 3 only provide the prestress loss at one specific point in time.

This paper summarizes an investigation of prestress losses measured in eleven Minnesota Department of Transportation (Mn/DOT) Type 28 prestressed concrete beams. All of the beams were subjected to flexural crack initiation and re-opening tests. All of the beams were instrumented internally with embedded vibrating wire strain gages in the concrete at the center of gravity of the steel, and two of the girders were destructively tested by gaging an exposed portion of the strands and subsequently severing the gaged strands. The investigation included an evaluation of the most effective means to determine the flexural crack initiation and re-opening loads.

**PREVIOUS RESEARCH ON PRESTRESS LOSSES**

Previous studies have been conducted to determine prestress losses in girders. Results of these studies are summarized in Table 1 and include field monitoring of girders under service and testing of newly-constructed girders or girders removed from existing bridges.

In the majority of cases involving field monitoring, Method 1 described above was used to determine the prestress losses. The girders were instrumented with vibrating wire strain gages embedded in the concrete at the level of the center of gravity of the prestressing strands and were subsequently monitored during construction and service life of the bridge.

Method 2 explained above is a standard procedure for determining the existing prestressing force in girders removed from service and girders constructed for the sole purpose of testing. In the latter case, researchers may use a combination of Methods 1 and 2. In published studies, flexural cracking and crack re-opening loads of girders were determined from visual observations or from the measurements of bottom surface strains or displacements. Some of the researchers also used Method 3 or another destructive testing technique.

As shown by the data available in the literature (Table 1), the measurements made using various methods can result in prestress losses that can be either smaller or larger than those predicted by available methods. Moreover, results from the previous studies also suggest that prestress losses measured in the same girder using different methods can differ from each other.

The three experimental techniques described above were used to determine the effective prestressing force in a series of Minnesota Department of Transportation (Mn/DOT) Type 28M prestressed concrete girders. This paper presents a description and comparison of the results of the techniques used to determine the prestress losses, as well as a description and comparison of the methods used to detect flexural crack initiation and re-opening loads during testing.

**EXPERIMENTAL STUDY**

The material presented in this paper was part of a study to investigate the effects of pre-release cracks on the performance of Mn/DOT Type 28M prestressed concrete bridge girders. Pre-release cracks are vertical cracks that develop during the fabrication of
prestressed bridge girders prior to the release of prestressing strands. The cracks initiate at the top surface of the girders and extend toward the bottom flange. Upon release, the cracks are often observed to close. Pre-release cracks in the test beams were artificially made in order to facilitate the placement of instrumentation and comparison of experimental and numerical results. The results regarding the effects of pre-release cracks are presented elsewhere; this paper is focused on the results associated with the determination of prestress losses and flexural crack initiation and re-opening loads.

**Description of Beams**

Table 2 lists the test specimens and their associated variables. The specimens consisted of eleven 20 ft. (6.1 m) long beams (Figure 1), three of which did not have pre-release cracks while the remaining eight incorporated a single pre-release crack at midspan with varying crack depths and crack widths see Reference 19 for details of the pre-release crack geometry. All beams were cast in a single pour with a nominal concrete strength of 8000 psi (55 MPa). Concrete strengths measured at the time of load testing of the beams are listed in Table 2.

Each beam had four straight 0.6 in. (15.2 mm) diameter seven-wire Grade 270 low-relaxation prestressing strands placed in one layer 2 in. (51 mm) from the bottom of the beams. The prestressing strands were initially tensioned to a nominal stress of 0.54\( f_{pu} \), where \( f_{pu} \) was the specified tensile strength of the prestressing steel. The low prestressing value was required because the beams were designed for ½ in. (12.7 mm) diameter strands with an initial prestress of 0.75\( f_{pu} \), but the strand diameter was changed to 0.6 in. (15.2 mm) during fabrication because of difficulties encountered in attaching vibrating wire strandmeters to the ½ in. (12.7 mm) diameter strands. In order to keep the prestressing force at the same level as the design value, the 0.6 in. (15.2 mm) diameter strands were only tensioned to 0.54\( f_{pu} \) during the fabrication of beams.

**Instrumentation**

Each beam had one Geokon VCE-4200 vibrating wire concrete strain gage embedded in the concrete during fabrication. The gages were placed between the strands (Figure 2a) at locations 18 in. (457 mm) away from midspan so that the gage readings would not be affected by the local strain changes caused by the pre-release cracks, which were located at the midspan. Because the moment due to the self weight of the beams was negligible, the losses measured by the vibrating wire gages could be assumed to be the same as the prestress losses at the midspan of the beams (the theoretical difference in prestress losses at the two locations was calculated as 10 psi [0.07 MPa]).

Strain and temperature readings from the vibrating wire gages were recorded during the fabrication of the beams and also before load testing.

The typical external instrumentation scheme is shown in Figure 3. The beams were heavily instrumented with concrete surface gages (with 2.36 in. [60 mm] gage length) in the vicinity of the pre-release cracks both through the depth and along the length of the beams. Typically, the strain gages were placed on the bottom surface of the beams at 2 in. (51 mm) spacing near midspan and the spacing was increased to 4 in. (102 mm) elsewhere in the constant moment region (Figure 2b).
Using gages with a relatively long gage length (2.36 in. [60mm]) minimized local effects due to aggregate size on the measured surface strains. Reliability of measurements from the concrete surface gages was verified by comparing the results to strain gages embedded in the concrete. In addition to the surface strain gages, electrical crack detection gages were mounted on one side of the bottom flange to detect the flexural crack initiation (Figure 2c).

LVDTs were placed at the bottom surface of the beams following the crack initiation tests. Measurements from these LVDTs, placed both over and next to existing flexural cracks, were used to determine the load at which the flexural cracks re-opened during the tests.

**Load-Testing of the Beams**

The beams were tested flexurally under four-point bending as shown in Figure 1. Elastomeric bearing pads supported the beams at the ends of the 19 ft (5.8 m) center-to-center span. Each beam was subjected to monotonically increasing load until flexural cracking occurred and the cracks became visible. Flexural crack re-opening tests were performed following the cracking tests to determine the bottom fiber “zero stress” loads. During the crack initiation tests, the beams were between 116 and 496 days old (Table 2).

Displacement-controlled loading was applied to all beams except beams SC-UC-1 and SC-22-14-R2, which were the last two beams tested. Under displacement control, the loads were initially applied at a rate of 0.025 in./min (0.635 mm/min). The loading rate was reduced to 0.0125 in./min (0.3175 mm/min) when the applied load approached the expected cracking load to facilitate visual flexural crack detection. For load-controlled tests, the loading rate was initially 3 kips/min (13.3 kN/min), which was reduced to 2 kips/min (8.9 kN/min) later in the tests.

**PRESTRESS LOSSES FROM VIBRATING WIRE GAGE DATA**

The first of the three experimental techniques to determine prestress losses in the Type 28M beams employed the embedded vibrating wire strain gage data. These data can be used to monitor prestress losses that occur throughout the life of the beams. In this study, the data was used to investigate prestress losses at release and at the time load testing. Assuming that the concrete perfectly bonds to the vibrating wire gage, the gage strain change should equal the change in concrete strain. Similarly, assuming perfect bond between the prestressing strands and the concrete, the change in concrete strain should equal the change in strand strain. Based on these principles, the reduction in the initial prestress in the strands could be determined.

Losses are defined as being the change in stress in the strand from the time the strands are tensioned until the time of interest. However, the vibrating wire strain gage cannot measure strain change in the concrete until the concrete has hardened, so it is necessary to consider three distinct times in order to back calculate the losses: $t_1$ – the tensioning time, $t_2$ – the time at which the strain gage is zeroed (after the concrete has hardened sufficiently, but before release), and $t_3$ – the time at which the losses are of interest. Although the strain in the strand is constant between times $t_1$ and $t_2$ because there is no change in the length of the strand prior to release, the stress in the strand is reducing because the strand temperature increases due to the heat of hydration of the concrete.
This change in strand stress between tensioning and zeroing of the strain gage needs to be taken into account.

The constitutive law for change of strand strain includes an elastic part, as well as two other parts: strain change due to relaxation, and strain change due to temperature change. As a result, the change in strand strain between the reference time and the current time was determined from

\[ \Delta \varepsilon_s = \frac{\Delta \sigma_s}{E_s} + \alpha_s \Delta T + \Delta \varepsilon_{\text{relaxation}}, \quad (1) \]

where \( \Delta \varepsilon_s \) is the change in strand strain occurring between the reference time and the current time; \( \Delta \sigma_s \) is the change in the prestressing strand stress (the loss); \( E_s \) is the strand modulus of elasticity; \( \alpha_s \) is the coefficient of thermal expansion of strand; \( \Delta T \) is the temperature change between the reference and the current times; and \( \Delta \varepsilon_{\text{relaxation}} \) is the change in strand strain due to relaxation.

Because the beams were lightly prestressed (each beam had only four 0.6 in. (15.2 mm) diameter strands initially tensioned to 54 percent of \( f_{pu} \)), prestress loss due to strand relaxation was negligible. As mentioned earlier, the strand stress change resulting from concrete temperature changes however, was not small enough to be neglected. Therefore, Equation (1), when solved for changes in strand stress, becomes

\[ \Delta \sigma_s = E_s (\Delta \varepsilon_s - \alpha_s \Delta T) \quad (2) \]

Because both \( t_1 \) and \( t_2 \) are prior to release, the strand strain (but not necessarily the strand stress) is the same at both times, i.e. \( \varepsilon_{s1} = \varepsilon_{s2} \). So, the change in total strand strain between \( t_3 \) and \( t_1 \) is the same as the change in total strand strain between \( t_3 \) and \( t_2 \)

\[ \Delta \varepsilon_{s3-1} = \Delta \varepsilon_{s3-2} \quad (3) \]

The equation used to convert the vibrating wire strain gage readings into a change in concrete strain is

\[ \Delta \varepsilon_{s3-2} = B (R_3 - R_2) + \alpha_g (T_3 - T_2) \quad (4) \]

where \( B \) is the gage calibration factor (specified to be 0.9752 by the gage manufacturer), \( R_2 \) and \( R_3 \) are the gage readings at times \( t_2 \) and \( t_3 \), respectively; \( \alpha_g \) is the coefficient of thermal expansion of the gage wire; and \( T_3 \) and \( T_2 \) are the concrete temperatures at times \( t_2 \) and \( t_3 \).

Assuming that the change in strand strain is equal to the change in concrete strain at the level of the center of gravity of strands, and combining Equations (3) and (4) gives

\[ \Delta \varepsilon_{s3-1} = B (R_3 - R_2) + \alpha_g (T_3 - T_2) \quad (5) \]

Inserting Equation (5) into Equation (2) and assuming that \( \alpha_g = \alpha_s \), the change in strand stress between times \( t_3 \) and \( t_1 \) becomes

\[ \Delta \sigma_{s3-1} = E_s [B (R_3 - R_2) - \alpha_s (T_2 - T_1)], \quad (6) \]
where $T_1$ is the steel temperature at time $t_1$. The term $E\alpha_s(T_2 - T_1)$ represents the locked-in strand stress change due to temperature changes that occurred between the time when the strands were tensioned and the vibrating wire gage readings were zeroed.

Total losses at the time of load testing of the beams were determined from the vibrating wire measurements using Equation (6), and are tabulated in Table 2. The coefficient of thermal expansion for strand, $\alpha_s$, was taken as 6.78 $\mu$ε/F (12.2 $\mu$ε/°C). The $T_1$ and $T_2$ values measured in the beams were in the ranges of 5.9-8.6 F and 31.3-40.1 F (3.3-4.8 C and 17.4-22.3 C), respectively. The strand modulus of elasticity was taken as 29,200 ksi (201,300 MPa) based on the total elongation measured during initial strand stressing.

As shown in Table 2, prestress losses due to elastic shortening were between 5.7 and 7.1 ksi (39 and 49 MPa). The total measured prestress losses at the time of load testing, (i.e., at age between 145 and 526 days) ranged between 20.7 and 26.4 ksi (143 and 182 MPa).

**PRESTRESS LOSSES FROM LOAD TESTING**

The second method of determining prestress losses was through load testing the beams. In this case, mechanics of materials was used to relate the effective prestressing force in the beams to the loads that caused initial flexural crack initiation and the loads that caused crack re-opening. Thus for beams with no pre-release cracks, there were two independent means of determining prestress losses from load tests. Determination of prestress losses using both the measured flexural crack initiation and crack re-opening loads requires accurate determination of the loads at which the flexural cracks initiate and re-open. In addition, the determination of prestress losses resulting from initial flexural crack initiation requires accurate knowledge of the concrete modulus of rupture. Material tests were used to determine the concrete modulus of rupture. To accurately determine the flexural cracking and crack re-opening loads of the beams, several methods were used. A description of each of these methods and comparison of their results are given below, followed by the determination of prestress losses using the flexural cracking and crack re-opening loads.

**Flexural Crack Initiation Loads**

Flexural crack initiation loads were determined using three methods: visual observation, crack detection gages, and strain gages placed at the bottom surface of the beams.

**Flexural Crack Initiation Loads from Visual Observation** - The loads at which the first flexural cracks were visually detected are listed in Table 3. Even though the beams were white-washed before the tests to make visual detection of the cracks easier and a careful inspection was performed continuously during the tests, the visual cracking loads documented in Table 3 represent an upper-bound for the flexural cracking loads in that the cracks must be wide enough to be visible by the naked eye. Similar observations were reported in References 6 and 20.

**Flexural Crack Initiation Loads from Crack Detection Gages** - Crack detection gages were the second means employed to determine flexural cracking loads. As shown in Table 3, flexural cracking loads indicated by crack detection gages were higher than the visual cracking loads for some cases and were lower for other cases. In beams SC-22-08 and SC-22-14-R1, flexural cracks were visually detected when loading was paused.
following the fracture of a crack detection gage. In beams SC-UC-2 and SC-UC-3, when
detection gages indicated flexural cracking, no crack was visually detected. Crack
detection gages in beams SC-16-08 and SC-16-14 indicated flexural cracking after a
flexural crack was visually observed. In beams SC-06-08, SC-06-14, and SC-22-14 crack
detection gages did not fracture before testing was terminated.

Because the cracking loads obtained by crack detection gages were not found to correlate
with any other means of monitoring crack initiation, their use was suspended after testing
the first nine beams. It was also reported in Reference 9 that crack detection gages were
not useful in determining crack initiation in prestressed concrete box girders.

Flexural Crack Initiation Loads from Concrete Surface Strain Gages - The flexural crack
initiation load of the beams was also determined by examining the variation of bottom
fiber tensile strain with the applied load. Figure 4 shows such a plot for two strain gages
(H4 and H5) placed at the bottom surface of beam SC-UC-3.

As shown in Figure 4, the strain readings of the two gages, placed 2 in. (51 mm) apart
from each other, started to diverge at 44 kips (195.7 kN). Starting at this load, gage H11
indicated constant strain with increasing load. This was due to tensile strain relief
resulting from flexural cracking occurring in the vicinity of the gage. In Reference 20 it
was reported that data from concrete surface gages placed at the bottom surface of a full-
scale post-tensioned concrete beam exhibited similar behavior at flexural crack initiation.
For all beams, except SC-16-08, the loads at which this type of behavior was observed
were determined, and are tabulated in Table 3. As shown in the table, flexural cracking
loads determined from the bottom fiber strain gages were smaller than the corresponding
visually observed cracking loads.

Discussion of Flexural Crack Initiation Loads - Among the three methods for determining
flexural cracking loads, the method of using bottom surface strain gage data was deemed
to be the most reliable. The visually determined cracking loads were always larger than
the flexural crack initiation loads determined using the bottom surface strain gages. The
visual loads were also observer-dependent and the results varied depending on the care
and frequency of the visual inspection performed during testing. The inconsistency
between the visually observed cracking loads of the three repeat beams (SC-UC-1, SC-
UC-2, SC-UC-3 and SC-22-14, SC-22-14-R1, SC-22-14-R2) as shown in Table 3 gives
an indication of observer-dependency. It should be noted that bottom surface strain gages
indicated consistent crack initiation loads for these repeat beams (except for SC-UC-2,
which was suspected to be flexurally cracked before the initial cracking test).

The crack detection gages did not provide a reliable means to determine crack initiation
loads. For several beams, the detection gages appeared to prematurely indicate crack
initiation due to straining of the concrete. For several other beams, the detection gages
did not indicate cracking even after the cracks became visible.

Flexural Crack Re-opening Loads
Following flexural crack initiation tests, the beams were unloaded and reloaded again to
determine the flexural crack re-opening loads. Different from crack initiation, crack re-
opening loads are independent of the tensile strength of the concrete. The cracks reopen
when the compression at the bottom fiber of the beam is overcome by the effect of
applied loading, and the stress at the location of the crack on the bottom of the beam becomes zero. The two types of instruments used to determine the crack re-opening loads were strain gages and LVDTs attached to the bottom surface.

**Flexural Crack Re-Opening Loads from Bottom Surface Strain Gages** - The method of using bottom surface strain gage readings to determine crack re-opening loads has been previously used by other researchers. The results of this method are tabulated in Table 3 for six of the beams in this study. For these beams, crack re-opening loads were taken as the loads at which there was a considerable change in the slope of load versus strain curves as shown in Figure 5. In some of the beams, the change in slope of the strain plots was not well defined and the re-opening loads could not be determined from the bottom fiber strain data. The reason that re-opening of the cracks could not be determined from the bottom fiber strain data for all of the beams, even though the gages had detected the initiation of the cracks, might be related to a change in order in which the flexural cracks at the bottom surface of the beams re-opened if there were multiple flexural cracks.

**Flexural Crack Re-Opening Loads from Bottom Surface LVDTs** – After flexural crack initiation tests, LVDTs were attached to the bottom surface of the beams over and adjacent to the flexural cracks that had the widest crack widths during the crack initiation testing. A sketch showing the location of these LVDTs with respect to major flexural cracks is shown in Figure 6a. These LVDTs showed bilinear behavior similar to that exhibited by the bottom surface strain gages. When the bottom fiber stress at the location of the flexural cracks reached zero during crack re-opening tests, the displacement measured by the LVDT placed over the crack started to increase at a faster rate because of crack opening. At the same time, displacement measured by the LVDT located next to the crack started to increase at a slower rate, because of the strain gradient in the vicinity of the crack. The load at which the initial slope of the load versus displacement plots changed was taken as the flexural crack re-opening load as shown in Figure 6b and are tabulated in Table 3.

Use of a similar technique to detect flexural crack re-opening in prestressed concrete beams was reported in References 6, 7, and 14. In Reference 8, crack re-opening loads of prestressed concrete beams were determined using crack opening measurements from clip gages placed across a flexural crack on one side of the specimen.

As shown in Table 3, flexural crack re-opening loads obtained by both the strain gages and LVDTs were in good agreement. In addition, both methods produced consistent results for the two sets of repeat beams (SC-UC-1, SC-UC-2, SC-UC-3 and SC-22-14, SC-22-14-R1, SC-22-14-R2).

**Prestress Losses from Measured Crack Initiation and Re-Opening Loads**
Because of the effects of pre-release cracks on beam strains, prestress losses from the measured flexural crack initiation and re-opening loads could not be determined for the beams incorporating pre-release cracks. For this reason, only the prestress losses for the beams without pre-release cracks (i.e., SC-UC-1, SC-UC-2, and SC-UC-3) were determined and compared to the losses determined by the other methods.

The effective prestress at the time of load testing of beams was computed using:
where, $f_r$, the concrete modulus of rupture, was taken as the measured value of 650 psi (4.48 MPa) for the case of crack initiation loads. In the case of crack-opening tests, the value of $f_r$ was taken as zero because the concrete was already flexurally cracked in the crack re-opening tests. The values of $A_{\text{gross}}$, $e_{\text{gross}}$, $S_{\text{gross}}$, $S_{\text{transformed}}$, and $M_{\text{self weight}}$ were 285 in.$^2$ (183,800 mm$^2$), 10.3 in. (262 mm), 1927 in.$^3$ (3.158*10$^7$ mm$^3$), 1992 in.$^3$ (3.264*10$^7$ mm$^3$), and 165 kips-in. (18.65 kN-m), respectively.

Using the average of the measured crack initiation loads of 46 kips (204.6 kN) from the bottom fiber strain gages for the uncracked beams SC-UC-1 and SC-UC-3, and the average of crack re-opening loads of 26 kips (116 kN) determined from the LVDTs attached on beams SC-UC-1, SC-UC-2, and SC-UC-3, the effective prestressing force, and consequently, the prestress losses could be predicted. The results are given in Table 4.

The reason for the discrepancy between the losses obtained using the crack initiation and crack re-opening loads [i.e., 98.4 and 69.2 ksi (678 and 477 MPa), respectively] was likely due to the value of concrete modulus of rupture assumed in the calculations for the flexural crack initiation case. Because prestress losses determined from crack re-opening loads are not dependent on concrete modulus of rupture, the prestress losses determined from re-opening loads are deemed to be more reliable. A concrete modulus of rupture of 420 psi (2.89 MPa) rather than the measured value of 650 psi value would be required to produce the same losses as obtained for the crack re-opening load case (69.2 ksi [477 MPa]). Considering potential variation in the measured modulus of rupture values, the fact that the modulus of rupture beams were not cured and kept in an identical environment as the beams, and the possible presence of restrained shrinkage at the bottom surface of the test beams, a value of 420 psi (2.89 MPa) for the concrete tensile stress at rupture is reasonable.

**PRESTRESS LOSSES FROM STRAND CUTTING TESTS**

Strand cutting tests were performed on two of the beams, SC-UC-3 and SC-22-14-R1, following the load tests. In each of the two beams, two of the strands were exposed over a short length by removing the concrete at the corners of the bottom flange on both sides of the beams. The exposed lengths of strand were approximately 20 in. (508 mm) at a distance approximately 50 in. (1270 mm) from the end of the beam. This distance was selected to be larger than the predicted transfer length of 30 in. (762 mm) so that the prestress at this location would be the full effective prestress. Because the self weight of the beams was negligibly small, the losses near the beams ends determined by the strand-cutting method could be assumed to be the same as the prestress losses at the midspan section of the beams. Calculations indicated that there should be a 90 psi (0.6 MPa) difference in strand stress between the locations of strand-cutting and the beam midspan.

Gages were bonded to three individual wires and oriented along the axis of the wires for each of the exposed strands. Care was taken to prevent any damage to the gages due to
heat and unwinding of the strands during cutting. The strands were flame-cut while data were collected. This procedure was used previously by other researchers.\textsuperscript{5,6}

Average strand strain changes obtained by the six gages in each beam were 3710 and 3700 $\mu$ε for beams SC-UC-3 and SC-22-14-R1, respectively, with corresponding coefficients of variation of 3.6 and 4.4%. The “apparent” modulus of elasticity of 31,100 ksi (214,400 MPa) measured along the individual wires during “in-air” strand tests was used to convert the measured strand strain changes to strand stresses. The higher value of modulus was expected, as the strain gages were aligned along the individual wire orientation, not along the axis of the strand. Based on the measured change in strand strain and apparent modulus of elasticity values, the effective prestress in beams SC-UC-3 and SC-22-14-R1 was determined to be 115.4 and 115.1 ksi (796 and 794 MPa), respectively.

The initial tensioning stress in the strands was 149.9 ksi (1034 MPa), determined by subtracting the loss due to seating from the initial jacking stress, which was obtained from the load cell attached to the hydraulic jack used to stress the strands. The total prestress losses that occurred between the fabrication of the beams and the time of the strand cutting tests was obtained as the difference between the initial tensioning stress and the effective prestress measured from the strand cutting tests. The total losses were calculated to be 34.5 and 34.8 ksi (238 and 240 MPa) for beams SC-UC-3 and SC-22-14-R1, respectively.

**COMPARISON OF PRESTRESS LOSSES OBTAINED BY EXPERIMENTAL METHODS AND PREDICTED BY PCI AND AASHTO METHODS**

Table 4 shows a comparison of the average losses determined for the initially uncracked Mn/DOT Type 28M beams (i.e., SC-UC-1 to 3) obtained from the three experimental techniques (i.e., vibrating wire gages, load testing, and strand cutting) along with the predicted losses computed according to the PCI Committee method\textsuperscript{11} and AASHTO LRFD method\textsuperscript{12} using measured material properties. For the PCI Committee method, the losses were computed at release and at the end of a one-year period, while the AASHTO method gave the losses at release and final long-term losses. As seen in Table 4, both the elastic shortening and long-term losses predicted by the two methods agree well with the losses determined from the vibrating wire gages.

Table 4 also shows that using the measured flexural crack initiation and re-opening loads to compute prestress losses resulted in gross overprediction of the losses compared to the other experimental and analytical methods. This observation suggests that there is a fundamental inadequacy in at least one of the assumptions used when calculating the losses using a strength of materials approach. When the mechanics of materials approach is used inconsistently with the data from the current experiments, i.e., using the visually observed cracking loads occurring at a bottom fiber stress of 420 psi (the adjusted modulus of rupture), then the prestress losses are predicted to be 12, 20, and 39 ksi for the uncracked beams SC-UC-1, SC-UC-2, and SC-UC-3, respectively. This observation suggests that Equation (7) better relates visually observed cracking loads with effective prestress, even though the assumptions used in this approach should relate the best estimate cracking loads (those determined with the strain gages) with effective prestress.
The inconsistency that exists in the use of Equation (7) for the beams tested in this study was found to exist in earlier studies, even though there is not a good discussion of this phenomenon in the literature. References 8 and 9 used a rearranged version of Eq. (7) to predict flexural cracking loads given the effective prestress and modulus of rupture. The data from these references support the results obtained in this study.

In the study documented in Reference 9, two 27-year-old 27 in. (686 mm) deep prestressed concrete box beams removed from an existing bridge were subjected to load tests. The predicted cracking load of the beams was determined to be 30.1 kips (133.9 kN) using the measured values of concrete strength and measured effective prestress. Effective prestress in the test beams were determined from the visually observed flexural crack re-opening loads. It was reported that flexural cracking was observed during load testing of one of the beams at a load of 29 kips (129 kN). The data from these references support the results obtained in this study.

Reference 8 presents the results of load tests performed on two 40-year old specimens, each including two 12 in. (305 mm) deep inverted T-beams with concrete fill in between and outside the beams. Flexural cracking was observed on one side of one of the specimens at a load of 16.5 kips (73.4 kN). Using the measured concrete strength and effective prestress values, the cracking load for this specimen was predicted as 16.4 kips (73 kN). Even though the predicted cracking load matches well with the visually observed cracking load, a plot of strain profiles documented for this specimens suggests that cracking occurred at a smaller load than the load at which the cracks became visible. The strain profile at a load of 10 kips (44.5 kN) shows signs of existence of a flexural crack on one side of the specimen 1 in. (2.5 mm) above the bottom.

In both of these cases, as in the current study, Equation (7) seemed to better relate visually observed cracking loads (as opposed to best estimate cracking loads) with measured prestress losses, even though the mechanics used to develop Equation (7) would suggest that the relation should correlate the very first indication of cracking with the measured prestress losses.

**CONCLUSIONS**

The following conclusions may be drawn from the material presented in this paper:

1. Visually observed flexural cracking loads provided an upper-bound for the measured cracking loads of the beams. Use of crack detection gages was not a reliable method of determining flexural crack initiation. Determination of flexural crack initiation loads using the data from bottom surface strain gages was deemed to be the most reliable method.
2. Use of strain gages or LVDTs mounted at the bottom surface of beams in the vicinity of flexural cracks was found to be an effective way of determining the crack re-opening loads.

3. The most effective ways of determining prestress losses in prestressed concrete beams are by (a) using vibrating wire gages embedded in concrete and (b) attaching strain gages to an exposed strand and severing the strand. In this study, total prestress losses of between 20.7 and 26.4 ksi (143 and 182 MPa) were determined from embedded concrete vibrating wire gage measurements. Strand cutting tests performed on two beams indicated a total prestress loss of 34.7 (239 MPa) ksi. Using the measured flexural crack initiation and re-opening loads did not result in reasonable losses.

4. Using the basic theory of mechanics with best estimates of measured cracking and crack re-opening loads overestimated the prestress losses of the beams tested in this study as compared to those determined from embedded vibrating wire gages and strand cutting. Conversely, even when the prestress losses are known accurately, the traditional theory of mechanics approach will overestimate crack initiation and crack re-opening loads. The data indicated that the equation used to compute flexural cracking loads yields load values that better represent the load at which the cracks become visible at the bottom surface of beams, rather than the loads that cause a bottom fiber stress equal to the concrete modulus of rupture. The same statement is valid for flexural crack re-opening loads as well. More research is needed to determine the reasons for this large discrepancy.

**REFERENCES**


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ACKNOWLEDGMENTS

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<table>
<thead>
<tr>
<th>Source</th>
<th>Structure</th>
<th>Method</th>
<th>Measured losses as a % of AASHTO predicted losses</th>
<th>Measured losses as a % of PCI predicted losses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ref 1</td>
<td>Field monitoring of four prestressed I-girders</td>
<td>VWE</td>
<td>63-67%&lt;sup&gt;12&lt;/sup&gt; (2 years)</td>
<td>---</td>
</tr>
<tr>
<td>Ref 2</td>
<td>Field monitoring of five prestressed I-girders</td>
<td>Vibrating wire gages on reinforcing bars embedded in concrete</td>
<td>35%&lt;sup&gt;12,13&lt;/sup&gt; (10 months)</td>
<td>---</td>
</tr>
<tr>
<td>Ref 3</td>
<td>Field monitoring of four prestressed bulb-tee girders</td>
<td>VWE</td>
<td>55%&lt;sup&gt;12&lt;/sup&gt; (5 months)</td>
<td>---</td>
</tr>
<tr>
<td>Ref 4</td>
<td>Field monitoring of seven prestressed girders in four different bridges</td>
<td>VWE</td>
<td>48-80%&lt;sup&gt;12&lt;/sup&gt; (1 year)</td>
<td>---</td>
</tr>
<tr>
<td>Ref 5</td>
<td>Field monitoring of twenty-six prestressed girders in two different bridges</td>
<td>VWE</td>
<td>50-80%&lt;sup&gt;12&lt;/sup&gt;</td>
<td>---</td>
</tr>
<tr>
<td>Ref 6</td>
<td>Lab testing of a 20-year-old prestressed box girder</td>
<td>FCRL</td>
<td>135%&lt;sup&gt;13&lt;/sup&gt;</td>
<td>143%</td>
</tr>
<tr>
<td>Ref 7</td>
<td>Lab testing of two 28-year-old prestressed girders</td>
<td>FCRL</td>
<td>55%&lt;sup&gt;13&lt;/sup&gt;</td>
<td>---</td>
</tr>
<tr>
<td>Ref 8</td>
<td>Lab testing of two 40-year-old prestressed specimens</td>
<td>FCL</td>
<td>72-85%&lt;sup&gt;13&lt;/sup&gt;</td>
<td>---</td>
</tr>
<tr>
<td>Ref 9</td>
<td>Lab testing of two 27-year-old prestressed box girders</td>
<td>FCL</td>
<td>---</td>
<td>50%</td>
</tr>
<tr>
<td>Ref 10</td>
<td>Lab testing of a 25-year-old prestressed I-girder</td>
<td>FCRL</td>
<td>50%&lt;sup&gt;13&lt;/sup&gt;</td>
<td>50%</td>
</tr>
<tr>
<td>Ref 14</td>
<td>Lab testing of two new prestressed I-girders</td>
<td>VWE</td>
<td>73%&lt;sup&gt;13&lt;/sup&gt; (375, 687 days)</td>
<td>91%</td>
</tr>
<tr>
<td>Ref 15</td>
<td>Lab testing of new prestressed bulb-tee girders</td>
<td>Embedded Carlson strain meters</td>
<td>43%&lt;sup&gt;14&lt;/sup&gt; (18 months)</td>
<td>---</td>
</tr>
</tbody>
</table>

*VWE = Embedded concrete vibrating wire gages; FCL = Flexural cracking loads; FCRL = Flexural crack re-opening loads. Note: 1 in. = 25.4 mm.
Table 2 — Specimen details and losses determined from vibrating wire measurements

<table>
<thead>
<tr>
<th>Beam designation</th>
<th>Pre-release crack?</th>
<th>Loss due to elastic shortening ksi</th>
<th>Total loss prior to load testing ksi</th>
<th>At time of load testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>SC-UC-1</td>
<td>No</td>
<td>6.7</td>
<td>23.4</td>
<td>Beam age days: 406</td>
</tr>
<tr>
<td>SC-UC-2</td>
<td>No</td>
<td>6.2</td>
<td>20.7</td>
<td>139</td>
</tr>
<tr>
<td>SC-UC-3†</td>
<td>No</td>
<td>5.7</td>
<td>21.5</td>
<td>116</td>
</tr>
<tr>
<td>SC-06-08</td>
<td>Yes</td>
<td>6.3</td>
<td>23.5</td>
<td>196</td>
</tr>
<tr>
<td>SC-06-14</td>
<td>Yes</td>
<td>6.5</td>
<td>21.6</td>
<td>208</td>
</tr>
<tr>
<td>SC-16-08</td>
<td>Yes</td>
<td>7.1</td>
<td>21.7</td>
<td>188</td>
</tr>
<tr>
<td>SC-16-14</td>
<td>Yes</td>
<td>7.1</td>
<td>21.9</td>
<td>235</td>
</tr>
<tr>
<td>SC-22-08</td>
<td>Yes</td>
<td>6.2</td>
<td>26.4</td>
<td>174</td>
</tr>
<tr>
<td>SC-22-14</td>
<td>Yes</td>
<td>6.1</td>
<td>23.1</td>
<td>154</td>
</tr>
<tr>
<td>SC-22-14-R1†</td>
<td>Yes</td>
<td>5.7</td>
<td>22.4</td>
<td>248</td>
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<tr>
<td>SC-22-14-R2</td>
<td>Yes</td>
<td>5.7</td>
<td>25.6</td>
<td>496</td>
</tr>
</tbody>
</table>

† Beams subjected to strand cutting tests.

Note: 1 in. = 25.4 mm; 1 psi = 0.00689 MPa.

Table 3 — Measured crack initiation and re-opening loads

<table>
<thead>
<tr>
<th>Beam designation</th>
<th>Visually observed cracking load</th>
<th>Cracking load from bottom fiber strains</th>
<th>Cracking load from detection gages</th>
<th>Max. load applied in cracking test</th>
<th>Crack re-opening load from bottom fiber strains</th>
<th>Crack re-opening load from LVDTs</th>
</tr>
</thead>
<tbody>
<tr>
<td>SC-UC-1†</td>
<td>67 kips</td>
<td>47 kips</td>
<td>no gages</td>
<td>67 kips</td>
<td>27 kips</td>
<td>27 kips</td>
</tr>
<tr>
<td>SC-UC-2†</td>
<td>64 kips</td>
<td>29 kips††</td>
<td>47 kips</td>
<td>64 kips</td>
<td>n/a††</td>
<td>26 kips</td>
</tr>
<tr>
<td>SC-UC-3†</td>
<td>57 kips</td>
<td>44 kips††</td>
<td>55 kips</td>
<td>60 kips</td>
<td>n/a††</td>
<td>26 kips</td>
</tr>
<tr>
<td>SC-06-08</td>
<td>40 kips</td>
<td>19 kips††</td>
<td>no crack</td>
<td>40 kips</td>
<td>10 kips</td>
<td>n/a††</td>
</tr>
<tr>
<td>SC-06-14</td>
<td>37 kips</td>
<td>26 kips††</td>
<td>no crack</td>
<td>40 kips</td>
<td>10 kips</td>
<td>10 kips</td>
</tr>
<tr>
<td>SC-16-08</td>
<td>25 kips</td>
<td>n/a†††</td>
<td>28 kips</td>
<td>28 kips</td>
<td>n/a†††</td>
<td>8 kips</td>
</tr>
<tr>
<td>SC-16-14</td>
<td>32 kips</td>
<td>15 kips††</td>
<td>39 kips</td>
<td>42 kips</td>
<td>6 kips</td>
<td>5 kips</td>
</tr>
<tr>
<td>SC-22-08</td>
<td>36 kips</td>
<td>19 kips††</td>
<td>36 kips</td>
<td>43 kips</td>
<td>3 kips</td>
<td>6 kips</td>
</tr>
<tr>
<td>SC-22-14</td>
<td>45 kips</td>
<td>12 kips††</td>
<td>no crack</td>
<td>46 kips</td>
<td>4 kips</td>
<td>4 kips</td>
</tr>
<tr>
<td>SC-22-14-R1†</td>
<td>31 kips</td>
<td>15 kips††</td>
<td>31 kips</td>
<td>35 kips</td>
<td>n/a†††</td>
<td>4 kips</td>
</tr>
<tr>
<td>SC-22-14-R2</td>
<td>35 kips</td>
<td>12 kips††</td>
<td>no gages</td>
<td>45 kips</td>
<td>n/a†††</td>
<td>4 kips</td>
</tr>
</tbody>
</table>

† Beams without pre-release cracks.
†† No cracking detected by the gages until the maximum applied load.
††† This beam was believed to have been flexurally cracked prior to testing.
†† † Flexural crack initiation or re-opening load for this beam could not be determined.

Note: 1 kip = 4.448 kN.

Table 4 — Comparison of measured and computed prestress losses

<table>
<thead>
<tr>
<th></th>
<th>Vibrating wire gages†</th>
<th>Strand cutting test</th>
<th>Flexural cracking load</th>
<th>Flexural crack re-opening load</th>
<th>PCI Committee</th>
<th>AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long-term</td>
<td>21.9 ksi</td>
<td>34.5 ksi</td>
<td>98.4 ksi</td>
<td>69.2 ksi</td>
<td>22.1 ksi††</td>
<td>24.6 ksi†††</td>
</tr>
<tr>
<td>Elastic shortening</td>
<td>6.2 ksi</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6.5 ksi</td>
<td>6.4 ksi</td>
</tr>
</tbody>
</table>

† Average of losses in beams with no pre-release cracks.
†† Predicted losses after one year.
††† Predicted final losses.

Note: 1 ksi = 6.89 MPa.
Figure 1 — Test setup. Note: 1 in. = 25.4 mm.

Figure 2 — Gauges used in beams; (a) vibrating wire concrete gauge to be embedded in concrete; (b) electrical resistance gauges placed at bottom surface of beams; (c) crack detection gauges.

Figure 3 — Typical instrumentation scheme.
Figure 4 — Flexural crack initiation indicated by strain readings in beam SC-UC-3. Note: 1 kip = 4.448 kN.

Figure 5 — Flexural crack re-opening indicated by strain readings in beam SC-16-14. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.
Figure 6 — Flexural crack re-opening indicated by LVDTs; (a) location of LVDTs at bottom surface of beams; (b) near-crack and over-crack LVDTs on beam SC-16-14. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.
Innovations in Prestressed Concrete Pavement

by D.K. Merritt and B.F. McCullough

Synopsis: Prestressed pavement is perhaps one of the most important yet underused innovations in prestressed concrete in the past century. Generally in the form of post-tensioning, prestress significantly reduces the required slab thickness and greatly improves pavement performance by reducing the occurrence of cracking. While the first reported use of prestressed concrete for pavements occurred in the late 1930s and early 1940s in Missouri, Michigan, and Maryland, the prestressing techniques common to today’s practices were first used in the 1940s in France. Domestic applications followed in 1953 at the U.S. Navy’s Patuxent River Naval Air Station. In 1971, the Federal Highway Administration (FHWA) initiated a new series of demonstration projects at Dulles International Airport; Hogestown, Pennsylvania; Brookhaven, Mississippi; and Tempe, Arizona. Later, in 1985, research at The University of Texas at Austin led the construction of a one-mile test section on Interstate 35 near Waco, Texas, which is still in excellent condition after 20 years. Despite the success of these and other more recent projects, prestressed pavement is still not widely used. New FHWA demonstration projects, however, are currently applying prestressed pavement technology to precast concrete pavement construction. Recent projects in Texas and California have demonstrated the viability of prestressed precast pavements for not only improving performance and reducing slab thickness, but also for expediting pavement construction.

Keywords: post-tensioned pavement; precast pavement; prestressed pavement
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**B. Frank McCullough** is the Adnan Abou-Ayyash Centennial Professor Emeritus in Civil Engineering at The University of Texas at Austin and former Director of the Center for Transportation Research. Dr. McCullough has a particularly strong interest and background in pavement design. During his career, Dr. McCullough has supervised over 50 research projects involving development of quality assurance and quality control specifications, planning, design, construction, rehabilitation, and maintenance of pavements.

**INTRODUCTION**

Prestressing takes advantage of concrete’s naturally high compressive strength. By inducing a compressive stress in the concrete, the flexural strength of the concrete is effectively increased. This is advantageous for pavement construction as pavements are normally designed around the concrete’s flexural strength.

Prestressed concrete pavement (PCP) is not a new concept, having a long evolution dating back to the 1930s. Approximately 16 km (10 miles) of prestressed pavement was constructed in Missouri, Michigan, and Maryland between 1937 and 1941 using a technique which compressed the pavement at early ages until the concrete had reached maturity. While this is not considered prestressed concrete in the classic sense (because the compressive stress was removed from the pavement after the concrete had matured), it still demonstrated the benefit of taking advantage of concrete’s inherent high compressive strength in order to minimize cracking (in this case, early-age cracking). Subsequent projects, most notably a 1.6 km (1 mile) section of prestressed pavement constructed in Texas in 1985, have demonstrated that with sound design and construction methods, prestressed concrete pavements can provide excellent long-term performance with minimal maintenance over the pavement’s life.

**SUMMARY OF PRESTRESSED CONCRETE PAVEMENT EXPERIENCE**

Prestressing techniques common to today’s practices were first utilized for pavement construction by Eugene Freyssinet for a runway at Orly Airport in France in 1943. Subsequent airport projects followed in Algeria, Austria, Belgium, England, France, Germany, the Netherlands, and New Zealand, with nearly 30.5 km (19 miles) of runway pavement constructed abroad prior to 1960. Highways applications were first demonstrated in France in 1946. Subsequent projects were constructed in Austria, Belgium, France, Germany, Great Britain, Italy, the Netherlands, Japan and Switzerland, totaling approximately 22.5 km (14 miles) of prestressed pavement prior to 1960.
Domestic applications of prestressed concrete pavement began with a 178 mm (7 inch) thick experimental pavement and the Patuxent River U.S. Naval Air Station in Maryland in 1953. Subsequent airfield applications were constructed by the U.S. Army Corps of Engineers and at San Antonio International Airport in the 1950s. Since 1960, only two other airfield pavements have been constructed with PCP, including a runway segment at Chicago’s O’Hare International Airport in 1980, and a section of taxiway at Greater Rockford Airport in Illinois in 1993. In total, just over 1.6 km (1 mile) of prestressed pavement has been used for airfield applications in the U.S.

U.S. application of prestressed concrete pavement for highways began with an experimental project in Pittsburgh, Pennsylvania in 1956. The first operational highway PCP was not constructed until 1971 near Milford, Delaware. The most notable highway applications to date, however, were constructed under FHWA research efforts, beginning in 1971. In 1971, a section of airport road at Dulles International Airport was constructed with PCP. This project led to the development of design recommendations for prestressed concrete pavement by ACI Committee 325. The success of the Dulles project led to subsequent FHWA-sponsored demonstration projects constructed in varying climates with varying traffic conditions. Between 1973 and 1977, additional PCP projects were constructed in Harrisburg, Pennsylvania; Brookhaven Mississippi; and Tempe, Arizona. The performance of these new demonstration projects has been varied, and all four have either been replaced or are flagged for replacement in the near future.

The varied success of these new demonstration projects, even at early ages, led to further investigation of the design and construction methods for PCP. This investigation, conducted at the Center for Transportation Research at The University of Texas at Austin, resulted in the construction of yet another demonstration project near Waco, Texas in 1985. For this project, several new design details and construction techniques were developed, resulting in a pavement which has required little maintenance over its life and is still in excellent condition 20 years after construction.

Between 1985 and 2002, the only other domestic highway application of PCP was a project constructed in Altoona, Pennsylvania in 1988. Although this pavement experienced significant cracking within the first two years after construction, it is still providing good ride quality and is still in service to-date.

Many different design features and construction techniques have been examined over the course of these FHWA highway PCP demonstration projects. Variations in slab thickness, slab length, slab width, base type, and prestressing characteristics have been explored. Each of these projects were constructed using the post-tensioning technique for prestressing, described below. Table 1 summarizes the characteristics of each of these projects constructed since 1971.

### TYPES OF PRESTRESSED CONCRETE PAVEMENT

There are essentially three types of prestressed concrete pavements that have been constructed in the U.S. and abroad: pretensioned, poststressed, and post-tensioned. Each
technique has advantages and disadvantages in terms of constructability. Post-tensioning, however, has been found to be the most practical and widely-used construction technique.

Pretensioning

Pretensioning involves anchoring stressing tendons, generally prestressing strand or wires, to fixed abutments at either end of the pavement slab, as shown in Figure 1. Prior to placing the concrete, these tendons are tensioned and the concrete is cast around the tendons. After the concrete has reached adequate strength to withstand the compressive stress from prestressing, the tendons are cut away from the abutments, transferring the stress in the tendons, through bond, into the concrete, putting the pavement slab in compression.

Some advantages of pretensioning include the elimination of external anchors (as compared to post-tensioning) and full bonding of the prestressing tendons to the concrete slab. Disadvantages of pretensioning are primarily related to constructability. Pretensioning requires either stressing abutments or “portable” stressing frames to be constructed on-site in order to tension the tendons. These abutments or frames can be very costly and labor intensive to construct, particularly for large paving projects where several miles of pavement are to be constructed.

Poststressing

Poststressing is a technique that involves compressing the pavement slab after it has hardened through external jacking. For poststressed pavement construction, long pavement slabs are constructed with short “leave-out” sections between the slabs and fixed abutments or anchor lugs at the extreme ends of the pavement, as shown in Figure 2. After the concrete has reached adequate strength to withstand the compressive stresses, jacking rams are placed in the leave-outs between pavement slabs and the pavement slabs are jacked against one another (and against the stressing abutments), putting the entire pavement in compression. When the desired level of compressive stress is reached, the leave-outs are filled with concrete to hold the slabs in place and the jacking rams are removed.

One major advantage of this paving technique is that prestressing tendons are not required, saving both material and labor costs. One of the main drawbacks to this technique, however, is that large, fixed abutments are still required at the ends of the pavement section, or at fixed intervals along the pavement, to prevent the slabs from simply pushing apart. If any movement occurs at these abutments over time, however, all of the compressive stress can be lost. Additionally, this technique requires careful attention to the stressing operation to ensure that buckling of the slabs does not occur.

Post-Tensioning

The most widely used prestressed paving technique is post-tensioning. Post-tensioning involves the application of a compressive stress to the pavement slab through tendons cast into the slab. The tendons can consist of post-tensioning bars, wire, or more commonly, 7-wire strand. These tendons are anchored to the external face of the slab, at
either end, using external anchors. After the concrete has matured to the required strength, the tendons are tensioned, transferring the compressive stress into the concrete through the anchors, as shown in Figure 3.

Either bonded or unbonded post-tensioning systems can be used. For an unbonded system, common practice is to use greased post-tensioning strands sheathed with either plastic or waterproof paper. These tendons are cast into the pavement slab and anchored at the face or ends of the slab. Tensioning of the tendons completes the stressing operation, leaving the strands permanently unbonded in the slab. One advantage of this system is that tendons can be replaced if they are accidentally cut or fail due to corrosion.

For a bonded system, hollow ducts (metal or plastic) are cast in the pavement slab during construction. After the concrete has been placed, the stressing tendons are threaded through the ducts in the slab and anchored at the ends of the slab through external anchors. The tendons are then tensioned, as before. Following the stressing operation, grout is pumped into the ducts to completely fill the space between the duct and the tendon, bonding the tendon to the slab. The primary advantage of a bonded system is that if a tendon is accidentally cut or fails due to corrosion, there is not a complete loss of prestress over the length of the tendon. Another advantage of bonded tendons is compatibility between the steel and concrete, which allows for the development of bond resistance at any point in the pavement as well as improved resistance to volumetric changes of the slab.¹

The main advantage of post-tensioning, which makes it the most viable technique, from the standpoint of constructability, is that it is essentially a “self-contained” system, not requiring temporary or fixed stressing abutments or frames. Each post-tensioned slab also acts independently of adjacent slabs, allowing for stressing of the different slabs at different times. Post-tensioning does still require careful attention of the timing of the stressing operation, however, and bonded post-tensioning systems require an additional grouting operation.

**BENEFITS OF PRESTRESSED CONCRETE PAVEMENT**

Portland cement concrete pavements are “designed” for cracking. This cracking is controlled either through joints or through reinforcement. For jointed concrete pavements (JCP), contraction joints are sawn into the slab at specific intervals to ensure that the pavement will crack at these joints. Dowel bars are often cast into the slab at these joints to provide load transfer between the slabs on either side of the joint. Although seldom constructed anymore, jointed reinforced concrete pavements (JRCP) have a mat of steel reinforcement or wire mesh cast into the slab between these contraction joints in order to arrest any cracks that may form between the joints. Continuously reinforced concrete pavements (CRCP) are constructed with a continuous mat of reinforcement over the length of the pavement and no contraction joints (only construction joints at the end of each day’s paving). The reinforcement percentage is specified such that the pavement will crack at regular intervals, normally 0.9-2.4 m (3-8
The reinforcement helps to hold these cracks closed tightly to prevent the intrusion of water and incompressibles.

Although conventional (non-prestressed) concrete pavements, if constructed properly, can easily provide 30 or more years of service, cracks (and joints) in the pavement inevitably become maintenance problems. Cracks can spall, due to freeze-thaw or aggregate bond problems, leading to a rough pavement surface. Cracks, and joints without load transfer devices such as dowels, can also lead to faulting of the pavement. If cracks become too close together and longitudinal cracks form near the transverse cracks, punching failure can occur. Additionally, cracks (and joints) provide an avenue for water to penetrate beneath the slab and into the base or subgrade. If proper drainage is not provided beneath the slab, this water can saturate the base or subgrade and lead to pumping, wherein the fine particles of the base or subgrade material are pumped out from under the slab, leaving voids beneath the pavement.

One of the major advantages of prestressed concrete pavement is minimizing or even eliminating cracks in the pavement slab. By inducing a precompressive stress in the slab, any stresses caused by wheel loads or environmental conditions must first overcome the compressive stress in the slab in order for cracks to form.

While joints can never be completely eliminated, prestressed concrete pavements minimize the number of these joints. Expansion joints are needed in order to “absorb” the expansion and contraction movements of prestressed pavement slabs, but these joints can be spaced as much as 366 m (1,200 ft) apart. The fewer the number of joints, however, the less the initial construction costs and long-term maintenance costs are likely to be.

Another major advantage of prestressed concrete pavement is reduced slab thickness. For a given pavement support structure (i.e. subgrade, subbase, base) and a given traffic loading condition, tensile stresses in a thicker pavement slab will be less than those in a thinner pavement slab. In general, the higher the stresses in a pavement slab, the faster it will wear out due to fatigue under repetitive traffic and environmental loading. Therefore, for a given pavement support structure, environment, and traffic characteristics, thicker pavements will generally last longer (withstand more traffic loading) than thinner pavements.

With prestressed concrete pavement, compressive stresses are induced in the slab over the life of the pavement (assuming prestress losses are properly accounted for and that the prestressing tendons remain intact). Therefore, the stress levels in a thinner pavement slab can be limited to the stress levels in a thicker pavement through the precompressive stress from prestressing. This permits the construction of much thinner prestressed concrete pavements. While a 300-355 mm (12-14 inch) thick pavement may be required for conventional (non-prestressed) pavement for a given support structure, a much thinner 150-200 mm (6-8 inch) prestressed pavement slab can be constructed by simply varying the prestress in the slab. This results in significant savings in materials and can mitigate problems with overhead clearance.
DESIGN CONSIDERATIONS

There are several unique considerations in the design of a prestressed concrete pavement. For the sake of brevity, and because they are the most commonly constructed type of prestressed pavement, only the design considerations for post-tensioned concrete pavements will be discussed herein.

Pavement support structure, traffic characteristics (i.e. volume and mix of vehicles), concrete properties, and environmental effects must be taken into account with the design of all pavements as they affect the thickness and expected design life of the pavement. Design considerations which require special attention for prestressed concrete pavement, however, include slab curing and warping, frictional resistance at the slab-base interface, prestress losses, and expansion/contraction and curling movements at the ends of the slab.

All concrete pavement slabs curl and warp due to temperature and moisture gradients over the depth of the slab. For example, if the top of the slab is warmer than the bottom, the ends of the slab will tend to curl downwards. Likewise, if the moisture content at the top of the slab is higher than at the bottom, the corners of the slab will tend to curl downwards. In prestressed concrete pavements these effects can be much greater due to the length of slab normally constructed. Slab curling and warping not only degrades the smoothness of the pavement, but also generates bending stresses in the pavement which can lead to cracking.

Frictional resistance at the slab-base interface, likewise, is accentuated in prestressed concrete pavements. Unlike conventional (non-prestressed pavements) with contraction joints generally spaced every 4.5-6 m (15-20 ft), prestressed pavements may have joints spaced as far apart as 366 m (1,200 ft). As pavement slabs expand and contract due to daily and seasonal temperature cycles, this movement is restrained by friction at the slab-base interface. This frictional resistance causes stresses to build up in the pavement particularly at mid-slab between the joints where movement of the slab is restricted the most. To reduce these frictional restraint stresses, a friction reducing or bond-breaker material is required beneath prestressed pavements to permit freer movement of the slabs.

Prestress losses are unique to prestressed pavements and must be accounted for in the design. Prestress losses include strand relaxation over the life of the pavement, creep and shrinkage of the pavement slab, seating/anchorage losses during post-tensioning, and friction and “wobble” between the post-tensioning strand and duct or sheath. Fortunately, there are many tools available for estimating these prestress losses based on decades of experience in the prestressed concrete industry.

Slab movements must also be taken into account with prestressed pavement design. Slab movements include both horizontal (expansion and contraction) movements, as well as vertical (curling and warping) movements. Because joints are spaced much further apart in prestressed pavements, they must accommodate much greater horizontal movement than conventional (non-prestressed) pavement slabs. The amount of horizontal slab movement at the ends is a function of the concrete properties (thermal expansion,
shrinkage, creep), the length and thickness of the slab, the amount of prestress in the slab, and the frictional resistance between the slab and base. As discussed below, the expansion joint detail used for the pavement must be able to accommodate these slab movements.

Vertical slab movement is most noticeable at the expansion joint, and can cause significant pavement roughness if not properly accounted for in the design. Vertical slab movement is affected also by the concrete properties and slab geometry, but also by temperature and moisture gradients over the depth of the slab and the stiffness of the base beneath the slab.

As mentioned previously, several different design characteristics were examined with the various prestressed pavement demonstration projects constructed in the U.S. Table 2 presents the design characteristics for these projects described previously.

CONSTRUCTION DETAILS AND METHODS

As with design, there are several unique aspects for construction of prestressed concrete pavements. Because post-tensioning is the most common PCP construction method, this discussion will focus on this technique.

Prestressing Details

Prestressing details are a critical aspect of PCP construction. The size and type of tendon used, layout, and anchorage details must all be considered. Post-tensioning tendons may be high tensile strength steel bars, or more commonly, 7-wire strand. The size of the tendon used, 13 mm vs. 15 mm (0.5-inch vs. 0.6-inch) strand, for example, can significantly affect the spacing of the tendons. For bonded systems, multiple-strand post-tensioning tendons can be used to reduce the number of tendons.

The layout of the prestressing system is also important. The most common prestressing system used is mono-strand longitudinal post-tensioning with gap slabs. Gap slabs are “leave-outs,” typically 0.9-1.5 m (3-5 ft) wide, between adjacent slabs where tendon stressing is completed, as shown in Figure 4. After the slabs on either side of the gap slab have been stressed, the gap slab is filled in. The main disadvantage to this technique is that the non-prestressed gap-slabs between the prestressed slabs may not hold up as well as the prestressed slabs. An innovative technique developed for the Waco, Texas, prestressed pavement project, however, utilizes “central stressing” for the post-tensioning operation. With this technique, pockets are cast into the pavement at the center of each slab. The post-tensioning tendons are anchored at the expansion joints and extend into these pockets where they are coupled together and stressed, as shown in Figures 5 and 6. This technique eliminates the need for gap slabs and permits a more continuous paving operation.

Transverse prestressing is an essential part of the prestressing system. Stresses caused by wheel loads on a pavement act in both the longitudinal and transverse directions and therefore prestress is needed in both directions for an adequate pavement design.
Investigation of the PCP projects in Mississippi, Pennsylvania, and at Dulles Airport found that all had longitudinal cracking. This cracking was attributed to inadequate transverse reinforcement or the lack of transverse prestressing. For this reason, transverse post-tensioning was incorporated into the Waco, Texas, PCP project. A unique system of looped transverse post-tensioning tendons was utilized, permitting stressing to be completed from blockouts rather than at the edge of the slab, as shown in Figure 5.

Joints

As mentioned previously, the transverse expansion joints must be able to accommodate the significant expansion and contraction movement of the pavement slab while also transferring wheel loads across the joint. Several different transverse joint details were developed for the various PCP projects constructed in the U.S. The four initial projects in Mississippi, Pennsylvania, Arizona, and at Dulles Airport all experience problems with at least some of the joints within a few years after construction. Analysis of these different joint designs led to the development of a unique armored expansion joint detail for the Waco, Texas, PCP project, as shown in Figure 7. This detail utilizes dowel bars for load transfer across the joint, and 0.6 m (2 ft) long deformed bars to tie the joint to the pavement. The neoprene seal can accommodate up to 100 mm (4 inches) of movement and prevents incompressible materials from entering the joint. The joints for the Texas PCP project are in excellent condition and continue to perform normally, requiring little to no maintenance after 20 years of service.

Longitudinal joints between adjacent pavement slabs should be located away from the traffic wheelpaths. If the full pavement width cannot be cast as a single slab, longitudinal joints should be located at the centerline and at the outside edge of the shoulder stripe. Whenever possible, shoulders should be cast monolithically with the traffic lane(s). When (transverse) post-tensioning tendons cross a longitudinal joint, provision should be made to ensure that the tendons are not severed by differential movement of the slabs on either side of the joint.

Friction Reducing Material

As discussed previously, prestressed pavement slabs require a friction-reducing material beneath the pavement slab to reduce the frictional-restraint stresses that build up during daily and seasonal temperature cycles. The material must be effective at reducing friction and it should be constructible and economical. Experience with previous PCP projects found polyethylene sheeting to best meet these requirements. A single or double layer of polyethylene has been used for various projects, and both have been found to be very effective.

Concrete Placement

Concrete can be placed with conventional slipform or fixed form paving equipment. It is important that the post-tensioning tendons or ducts are properly secured to prevent movement during concrete placement. The tendons or ducts should be as straight as possible to minimize “wobble” losses during the post-tensioning process. If unbonded tendons are used, it may be necessary to apply a minimal amount of tension to the
tendons to straighten them out prior to concrete placement. Special care must be taken when paving over blockouts or leave-outs to keep from disturbing them and to ensure the smoothest surface possible.

**Stressing Sequence**

The stressing sequence is critical for ensuring that no early-age cracking occurs in the prestressed pavement slab. A minimum of two stages of post-tensioning is recommended. The first stage should be completed within 8-12 hours after placement of the concrete. This first stage of stressing helps to prevent the formation of early-age cracking as the slab contracts during the first cooling cycle after hydration of the cement. The exact timing of the first stage is specific to the mix used for the pavement, but should only be enough to prevent tensile stresses in the slab, and should be limited such that it will not cause localized cracking in the anchor region. The magnitude of the initial prestress should be selected based on length and thickness of the pavement slab, the frictional characteristics of the slab-base interface, and the spacing of the prestressing tendons. Concrete strength should be carefully monitored to determine the optimal timing of initial stressing.

The second stage of post-tensioning should apply the full and final post-tensioning force to the slab (unless a third stage is utilized). The second stage should only be completed after the concrete has reached adequate strength to withstand the compressive stresses without localized anchor failure. Normally the second stage is completed 2-3 days after construction, although this is highly dependent upon the concrete mix and curing conditions.

**PRECAST PRESTRESSED CONCRETE PAVEMENT**

Building on the successes of the cast-in-place prestressed pavement demonstration projects, particularly the Waco, Texas PCP project, for which several innovative construction techniques were developed, the FHWA initiated a research study in 1998 to evaluate the feasibility of precast prestressed concrete pavements. As state highway agencies are continually faced with shorter and shorter windows for reconstructing pavements in urban areas, new and innovative techniques are continually being sought for expedited pavement construction.

Precast concrete has a proven track record in the building and bridge industries for providing a rapid construction solution of a durable, high-performance product. Applying this knowledge to the pavement industry is only a logical step. Precast concrete eliminates many of the problems commonly associated with cast-in-place pavement construction, such as built-in curl (from temperature and moisture gradients in the slab), surface strength loss (from insufficient curing), and inadequate air entrainment. These benefits will greatly enhance the performance of precast pavements.

Following the FHWA feasibility study, completed at the Center for Transportation Research at The University of Texas at Austin, an initial demonstration project was constructed on the frontage road of Interstate 35 near Georgetown, Texas in 2002.
Approximately 0.7 km (2,300 ft) of precast prestressed concrete pavement was constructed, spanning the full-width of the frontage road. The precast panels were all pretensioned in the transverse direction during fabrication in order to provide the required transverse prestress, and post-tensioned together in 76 m (250 ft) sections in the longitudinal direction using the central stressing technique. Figures 8 and 9 show the construction process and finished pavement for the Texas precast pavement.

This initial project demonstrated the viability of not only prestressed pavement construction, but the application of prestressed pavement technology to precast pavement construction. Following this project, another FHWA-sponsored demonstration project was constructed on Interstate 10 in El Monte, California in 2004. Approximately 76 m (250 ft) of precast pavement was installed and post-tensioned over the course of two days. The pavement was post-tensioned in two separate 38 m (125 ft) sections. Installation of the precast panels was limited to nighttime hours to minimize disruption to traffic. This not only provided the California Department of Transportation (Caltrans) with a rapid construction solution, but perhaps more importantly, a long-term solution. Figures 10 and 11 show the construction and finished pavement slab for the California precast pavement.

CONCLUSIONS

Prestressed concrete pavement is not a new idea. Numerous projects in the U.S. and abroad, described above, have demonstrated the viability and benefits of this technique for pavement construction. Further, the commercial building and housing industries routinely use prestressing for building foundations and house slabs. Despite this fact, prestressed pavement construction has only seen very limited use by the highway industry.

With the nation’s pavement infrastructure continuing to rapidly deteriorate, and the demand for new or expanded highways continuing to increase, new techniques must be utilized for ensuring long-term pavement performance. Prestressed concrete pavement, while limited in use, has been demonstrated as a technique to provide this long-term performance. It has also been applied to precast pavement technology, which further provides an expedited construction solution. It is hoped that the proven performance and proven construction techniques developed over the years will make this a more widely used construction technique in the future.

REFERENCES


Merritt and McCullough


(4) Diaz, M.; Alberto, B.; McCullough, F.; and Burns, N.H.; Design of the Texas Prestressed Concrete Pavement Overlays in Cooke and McLennan Counties and Construction of the McLennan County Project, Research Report No. 555/556-1, Center for Transportation Research, The University of Texas at Austin, February 1986.


Table 1 – Design characteristics for prestressed pavement highway projects in the U.S.²

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<thead>
<tr>
<th>State</th>
<th>Virginia</th>
<th>Pennsylvania1</th>
<th>Mississippi</th>
<th>Arizona</th>
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<tbody>
<tr>
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<td>Waco</td>
<td>Altoona</td>
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<td>4</td>
<td>4</td>
<td>2</td>
<td>2</td>
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<td>137</td>
<td>122</td>
<td>73, 134</td>
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<td>26</td>
<td>58</td>
<td>30</td>
<td>18 @ 73 m</td>
<td>14 @ 134 m</td>
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<td>Slab Thickness, mm</td>
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<td>152</td>
<td>152</td>
<td>152</td>
<td>152 (overlay)</td>
<td></td>
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<tr>
<td>Base</td>
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Table 2 – Prestressing characteristics for prestressed pavement highway projects in the U.S.²

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<th>Arizona</th>
<th>Texas</th>
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<td>7.3</td>
<td>7.3</td>
<td>9.6</td>
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<td>Slab Length, m</td>
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<td>183</td>
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<tr>
<td>Slab Thickness, mm</td>
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<td>152</td>
<td>152</td>
<td>152</td>
<td>152 (overlay)</td>
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<td>No. of Tendons</td>
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<td>12</td>
<td>12</td>
<td>12</td>
<td>16</td>
<td>15</td>
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<td>610</td>
<td>610</td>
<td>610</td>
<td>610 for 73 m slabs</td>
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<td>Tendon Diameter, mm</td>
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<td>62.3 kN @ 6900 kPa</td>
<td>62.3 kN @ 6900 kPa</td>
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<td>variable-up-to 73.4 kN</td>
<td>69.4 kN @ 6900 kPa</td>
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<td>1,580</td>
<td>1,484</td>
<td>2,222 for 73 m slabs</td>
<td>2,567</td>
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²: Virginia, Pennsylvania1, Mississippi, Arizona, Texas, Pennsylvania2
Figure 1 – Schematic diagram of pretensioned pavement construction.1

Figure 2 – Schematic diagram of poststressed pavement construction.2

Figure 3 – Schematic diagram of post-tensioned pavement construction.3
Figure 4 – Diagram of a gap-slab used for post-tensioned pavement construction.¹

Figure 5 – Diagram the prestressing tendon layout for the Texas PCP project. Note the central stressing pockets for longitudinal post-tensioning and the looped tendons for transverse post-tensioning.⁴
Figure 6 – Schematic of the coupler used in the central stressing pockets for the post-tensioning tendons.  

Figure 7 – Armored expansion joint detail developed for the Texas PCP project.
Figure 8 – Placement of a precast panel for the Georgetown, Texas, precast prestressed concrete pavement. Note the central stressing pockets in the precast panel.

Figure 9 – Precast prestressed concrete pavement near Georgetown, Texas, after opening to traffic.
Figure 10 – Installation of a precast panel for the California precast prestressed concrete pavement.

Figure 11 – Finished section of the California precast prestressed concrete pavement just prior to post-tensioning.
Advances in Post-Tensioned Parking Facilities Seismic Design

by M. Iqbal

Synopsis: Post-tensioned concrete is a popular material used in the construction of parking facilities due to economy and durability it offers. The flooring system in post-tensioned parking facilities generally consists of one-way post-tensioned slabs supported by post-tensioned beams cast monolithic with concrete columns form moment frames. However, the building codes have restricted the use of post-tensioned concrete as primary lateral load systems in high seismic regions due to concerns about ductility, anchorage, bond, transfer lengths, grouting, characteristics of high strength prestressing steel and level of prestressing. Until recently, building codes have insisted on the ductile non-prestressed steel as the sole reinforcement and have not permitted the use of prestressing in ductile moment frames. This paper summarizes issues, advances and historical developments of code requirements concerning the prestressed or post-tensioned frames, followed by performance of parking facilities during recent major earthquakes. Next, the paper provides an overview of the current state of seismic design process for post-tensioned parking facilities and its structural systems, subsystems and elements.

Keywords: concrete; design; earthquake; garage; parking; post-tension; prestress; seismic; structure
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**Research Significance**

Post-tensioned concrete is a widely used in construction of parking facilities; however, its use to resist lateral loads in high seismic regions has been very restricted. This affects the cost of construction and burdens the economy. This paper summarizes the state of knowledge in design of the parking facilities and their performance in recent earthquakes and points out areas needing further research. Despite the progress made in the areas of seismic design in general, there is little focus on the design of parking facilities. This paper presents a unique perspective in order to achieve an optimal design.

**INTRODUCTION**

Concrete has been the most popular material used in construction of parking facilities in the United States. The post-tensioned (P/T) concrete has dominated the market due to economy, durability and monolithicity it offers, with second runner-up being the precast concrete. Post-tensioned parking facilities are generally stand-alone structures, rectangular in plan and one to twelve levels of supported parking. Figure 1 shows a post-tensioned parking structure with 949 car stalls. Figure 2 shows the largest post-tensioned structures in the World located in high seismic zone with over 10,000 car parking stalls and additional bus and RV parking areas on the ground floor.

During the 1994 Northridge earthquake, several parking facilities were destroyed or suffered disproportionately heavy damage. The parking facilities affected by the earthquake were of concrete using precast, post-tensioned, or hybrid structural systems. The scale of damage put new focus on design of the parking facilities in high seismic regions. A great deal of time and effort has gone into understanding the behavior of post-tensioned concrete members and structures under strong ground motions and towards development of better methods of design.

In designing for seismic resistance, safety of the occupants is the primary concern. A secondary concern is economics. The current building code philosophy is to prevent non-structural damage in minor earthquakes, allow some repairable damage in moderate earthquakes and prevent total collapse in major earthquakes. This paper presents a summary of recent developments in parking structure design along with the historic background, with emphasis on unbonded post-tensioned concrete structures. Structural behavior during recent earthquakes and the causes of distress are discussed first, followed by a commentary on design of the structural systems, subsystems and elements commonly used in various seismic regions. At the end, case studies of several parking facilities recently designed and built in seismic regions are presented.
The discussion is focused on post-tensioned structures with unbonded tendons because where prestressed members are used as primary lateral load resisting elements they are likely to be post-tensioned. Second, the post-tensioned parking structures use unbonded tendons more frequently than bonded tendons in the United States. The post-tensioned structures with shear walls are not discussed as their behavior is very close to their counterpart—cast-in-place [CIP] structures. The discussion below is limited to structures in which post-tensioned moment frames with post-tensioned flooring system and CIP columns are used as the primary earthquake resistant system.

HISTORICAL DEVELOPMENTS

Over the past 40 years, several state-of-knowledge reviews on the design of earthquake resistant prestressed concrete structures have been published. Hawkins reviewed and summarized then available analytical and experimental studies on seismic resistance of prestressed concrete elements and sub-assemblages. Most recently, the ACI-423 has published state-of-the-knowledge reports on prestressed and post-tensioned concrete design. For parking structures, References 1 and 6 provide a state-of-the-art review and commentary on the analysis and design methods used in the United States at dawn of the 21st century.

It is well known that the difference between responses of post-tensioned and reinforced concrete sections under seismic loading does not lie in the presence of the prestressing or pre-compression force per se, but in the stress-strain characteristics of the steel reinforcement usually used in the two sections. The Grade 60 reinforcing bar can stretch up to 15 percent strain, but the high strength strands or tendons usually have a strain of 4 to 6 percent at fracture over a specified gauge length. Because of the differences in the steel types used as reinforcement, there is a large difference between the energy dissipation characteristics of the two sections, with post-tensioned section dissipating much less energy than the reinforced concrete section. Because of this limitation, the Building Codes permitted only certain building systems in high seismic regions. The codes specified that the reinforcing steel in the elements that are a part of the primary seismic load resisting system in high seismic zones should be ASTM A706 steel with a yield stress of 60,000 psi (413 MPa). This requirement excluded precast-prestressed systems from being used in high seismic zones. The unbonded and bonded post-tensioned concrete systems, despite being inherently monolithic, were also excluded. This was a death knell for post-tensioned concrete construction as the primary lateral load resisting system in high seismic zones.

In 1988, Hawkins and Ishizuka tested sub-assemblages, and based on the test results, questioned the building code provisions requiring that the seismic forces to be resisted entirely by reinforcement with a yield strength not greater than 60,000 psi (460 MPa). In light of the test results, it was concluded that a limited amount of prestress does not adversely affect the response of ductile moment resisting frames. A maximum of 350 psi (2.5 MPa) prestress based on the beam’s rectangular cross-sectional area was recommended for flexural elements. The recommendation was first codified in BOCA
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1999 code. With some modifications, its predecessor, IBC 2000 and IBC 2003 codes have continued allowing partial pre-stressing in the seismic load resisting frame system in high seismic zones. The building codes’ restrictions on the use of prestressed or post-tensioned concrete elements have provided a challenge and opportunity for the industry to develop new systems. A discussion on emerging lateral load resisting systems that combine attributes of precasting and post-tensioning is beyond the scope of this paper.

STRUCTURAL BEHAVIOR DURING EARTHQUAKES

Earthquake engineering is experience-driven. Earthquakes reveal the shortcomings and deficiencies in the constructed facilities, and the engineering profession modify design and constructions requirement in order to minimize the loss from recurring.

In general, the causes of damage or collapse of structures include incomplete or poor load path, strong beams and weak columns, soft (weak) first stories, poor layout of structural walls, inadequate spacing between adjacent structures, column shear failures, inadequate confinement of column core, biaxial loading, particularly at corner columns, failure of beams and beam-columns connections, and failures of structural walls. For unbonded post-tensioned structure, reliability of end anchorages, and energy dissipation are additional concerns.

In 1970’s, Hawkins studied the behavior of post-tensioned structures in three earthquakes, namely, the 1970 Managua, 1971 San Francisco, and 1968 Tokachioki, Japan earthquakes, and concluded that most prestressed concrete elements, when designed for loading reversal, perform well in earthquakes, and failure that occurred were mainly due to failures of the supporting structures or connections. Hawkins noted that the prestressed beams show elastic recoveries after having undergone considerable inelastic deformations and subsequently have less residual damage and permanent deformations than that accompany non-prestressed elements surviving a major earthquake.

Loma Prieta Earthquake

After 1989 Loma Prieta earthquake, Aalami et al surveyed over 200 post-tensioned buildings in San Francisco area. The primary finding of this survey was that there was no notable damage, such as blow outs, rupture of strands, cracking or dislodging of anchorage in post-tensioning, as a result of the earthquake. This meant a significant improvement in performance of post-tensioned structures over the Alaska earthquake of 1964, during which several anchorages of post-tensioned floors failed resulting in loss of post-tensioning. However, during the Loma Prieta earthquake, post-tensioned slabs performed very well as rigid diaphragms in transferring lateral loads to seismic lateral load resisting elements such as shear walls or moment frames. The improved performance of post-tensioned flooring system was contributed to the stringent building
code specifications added after the previous earthquakes’ experience. Aalami et al. further noted that hybrid or multi-use structures also performed well during the Loma Prieta earthquake. These structures included parking facilities having residential or commercial improvements above parking levels, with walls and columns misaligned from one use to the next. Based on the performance of post-tensioned floor systems, it was suggested that though the buildings did not utilize post-tensioned moment frames in resisting seismic shear, the post-tensioned lateral load resisting systems have the potential to exhibit superior performance during major earthquakes.

**Northridge Earthquake**

The 1994 Northridge earthquake was the next severe earthquake that struck California causing severe damage in the area. Hundreds of post-tensioned concrete buildings were located in the severe damage zones and were investigated for damage. It was concluded that no failure or distress of any cast-in-place post-tensioned building designed using either 1976 or post-1976 code was attributed to post-tensioning. It was noted that despite the fact that no structure failed because of post-tensioning, a number of post-tensioned structures failed because their lateral system was inadequate. This included several parking facilities that were destroyed or suffered disproportionately heavy damage. Figures 3 thru 7 depict some damage to parking facilities caused by the Northridge earthquake.

Figure 3 shows an aerial view of the collapsed California State University [CSU], Northridge, parking facility. This facility used a hybrid system wherein precast concrete moment frames were slated to resist seismic lateral forces. For gravity loads, post-tensioned slab acting compositely with precast beams was used. The precast beams were supported on precast column corbels. During the earthquake, some of the beams slid off the corbels supports, triggering the diaphragm to collapse. Fig. 4 shows a precast beam-column joint with beams in a precarious state after having rotated and slid on the corbel supports following the supporting column failure. Figure 5 shows a close-up view of the subsequential premature collapse of the perimeter lateral load-resisting system caused by the floor diaphragm separating itself from the lateral load-resisting frames. Several corbels seen in photo show the locations where beams were once seated on the frame. At the time of collapse, the CSU facility was no more that two-year old, presumably built in conformance with the modern building code. It is worth noting that a post-tensioned located less than half a mile from the collapsed CSU facility sustained no damage. Fig. 6 shows a parking facility in which several columns along the perimeter failed in shear. Here, the columns were cast integrally with the security wall on the first floor and with bumper wall on the second floor, reducing the column clear height tremendously. As shown in Figure 7 close-up view, the perimeter columns had very few ties. The columns were short and relatively stiff and, therefore, attracted a greater portion of the seismic input and required generation of large shear forces to develop the moment capacity of the column section. As the shear strength of the columns was less than their respective flexural capacity, the columns failed in brittle shear, rendering building irreparable.
For post-tensioned structures, integrity of the anchorage devices is of concern. During Northridge earthquake, the anchorage system remained intact in all cases except that in CSU facility certain wedges broke loose during the ground motion and released the tendons. Aalami pointed out that the anchors were not cast in concrete as is normally done, but were cast against the precast panels using a non-conventional detail.

The lessons learned from the Northridge earthquake were codified in the next version of the building code, namely, UBC-97. The code, among other things, prescribed several measures to enhance structural redundancy and floor diaphragm stability and performance.

**Nisqually Earthquake**

The next major earthquake to hit the west coast after the Northridge earthquake was the Nisqually earthquake. It hit Seattle, Washington area on February 28, 2001. The earthquake had a Richter magnitude of 6.8 and occurred 11 miles (18 Km) NE of Olympia, WA at the hypocentral depth of 32 miles (51 Km). The peak ground acceleration in SeaTac Airport area was about 0.1g which is about one-half the intensity of a design earthquake. Despite the moderate intensity of the earthquake, the loss of property in Seattle area was in billions. On the other hand, the post-tensioned parking facilities in Seattle area exhibited little structural damage.

The SeaTac International Airport parking facilities consists of five parking structures adjacent to one another, built in phases from 1970’s to 1999. These structures have seven supported levels of parking. Eight single-threaded helices provide access to and egress from the parking areas. Structurally, the helices are independent cantilevers. As shown in Figure 8, the helices offer a unique architectural expression. During the post-earthquake survey, it was noted that the relative displacement between a helix and the adjacent structure during the earthquake exceeded 4 inches (100 mm). Even with the swing this large, the helices returned to the original position or very close to it, with plumbness within acceptable tolerance for a serviceable structure.

Among the five parking structures, the newly completed section using post-tensioned beams and slabs, and CIP ductile frames exhibited the least amount of distress. This exemplary behavior was primarily because of the compliance to requirements of governing code, namely, UBC-97, a more stringent code than its predecessors. The only distress noted was at ground level where a few columns exhibited concrete cover spalling in the pour strip region.

**DESIGN REQUIREMENTS**

Parking facilities are utility structures and the “form follows function” axiom is widely used in selection of both the structural and/or lateral systems. The selection of a lateral load resisting system is strongly influenced by parking layout, internal flow pattern, volume change, openness requirements, durability and the intensity and types of
lateral loading. Seismic loading depends upon the site where the parking facility is located.

In designing a structure to withstand earthquake forces, the building codes have traditionally allowed lower lateral forces than those anticipated keeping the structure within the elastic limits implying that the structure would be able to deform inelastically, exhibiting ductility and energy dissipation. Although code seismic coefficients have undergone a series of major revisions in the last thirty-some years, the code philosophy has remained unchanged. Ductility is important in seismic design because it allows the structure to dissipate energy and redistribute forces. It insures a gradual rather than a brittle failure and provides a warning to occupants before collapse. Factors affecting ductility include the yield strength of reinforcing steel, longitudinal reinforcement index, transverse reinforcement, concrete strength, concrete core confinement and shear stresses on the section.

For post-tensioned parking structures, the effects of anchorage, bond, transfer lengths, grouting, characteristics of high prestressing steels and level of prestressing are additional parameters affecting ductility. Traditionally, the building codes have allowed post-tensioned frames in low to moderate seismic regions, but have insisted that in high seismic zones ductile non-prestressed steel be used as the sole reinforcement, disallowing post-tensioning in ductile moment frames. The following sections discuss the structural elements and subsystems design requirements and issues as applicable to the design of post-tensioned parking facilities.

Floor and Ramp Systems

In general, post-tensioned parking structures are rectangular in plan, with each level having two or more bays, out of which some are flat and others are sloping or ramping. Figure 9 shows a four-bay structure with two flat bays and two ramping bays sloping in opposite directions. In facilities where speed ramps or helices are used for ingress and egress of the vehicles, all bays can be flat. Figures 1 and 8 illustrate helices.

The most common floor and ramp system consists of one-way slabs. The slab is cast integrally with beams and is supported on the beams. The beams are supported on columns or on girders that, in turn, are supported on columns, as illustrated in Figure 10. In general, slabs, beams, and girders are of post-tensioned concrete, and the columns are of CIP concrete. Pour strips are used to isolate the slabs from the shear walls during post-tensioning in order to let the post-tensioning force precompress the slabs. Similarly, where column stiffness is such that it causes excessive loss of post-tensioning force or undesired cracking in the beams, vertical pour strips are introduced in columns to let the column flex during post-tensioning process. One way to create the vertical pourstrip in columns is to have a temporary hinge at the column base during construction, as shown in Figure 11. After the post-tensioning operation is complete and early shrinkage period has gone by, the temporary hinge is packed with concrete so the column attains its full section with design strength and stiffness, as shown in Figure 10.
Several computer programs are available to design post-tensioned slabs and beams, among which ADAPT software seems to be versatile and is widely used. With the use of software, the design of post-tensioned slabs and beams is a routine design task and will not be discussed here.

An unresolved issue in design of post-tensioned T-beams is the flange width that can be used in analysis and design. The ACI-318 does not distinguish between the flange widths of a cast-in-place T-beam and a post-tensioned T-beam. The flange width expression \(16t + b\) in the ACI-318 is to account primarily for the shear lag phenomenon. However, the behavior of the post-tensioned T-beam is different in that pre-compression force is applied to the post-tensioned beam at tendon’s anchorage. The force distributes itself into the flange as it travels from the anchor block and becomes practically uniform across the entire slab-beam flooring. For slab-beam layout commonly used in parking facilities, \(45^\circ\) dispersion is considered a conservative approximation. Using this approach, the wedge regions require local mild steel reinforcement for shrinkage crack control. Additional studies are needed to determine whether the ACI flange width approach needs modification for post-tensioned beams.

**Diaphragms**

The slab and beam flooring system provides a shear diaphragm which is suitable for use in facilities located in all seismic zones. The post-tensioned concrete structures are commonly analyzed for lateral loads assuming that the floor system acts as a diaphragm, is infinitely stiff in its own plane and distributes horizontal forces to the lateral load-resisting elements. Whether or not a diaphragm can be assumed to be rigid depends on several factors, such as span-to-depth ratio of the slab, plan dimensions relative to the location of the lateral load-resisting elements, slab thickness, locations of openings and discontinuities in the slab. For post-tensioned parking facilities, the assumption is generally considered valid, particularly in the light of the fact that the building codes permit the use of pre-stressing tendons in resisting diaphragm forces and limit the tendon stresses in order to help keep the diaphragm crack free.

When a parking structure has one or more sloping ramps, the diaphragm can be divided into several sub-diaphragms. Figure 12 shows a plan for a three-bay parking facility in which two bays are flat and the third is sloping. The diaphragm consists of three sub-diaphragms for the lateral load in the direction shown. Each sub-diaphragm should be designed separately so that it can resist its share of the load. The Building Code approach to designing floor diaphragms is quite simple and is described in several texts. In certain instances, where the load flow is not straight-forward, the finite-element method is used in analysis and design of a floor diaphragm.

**Ramp Trusses**

In a continuous ramped parking structure where floors slope and form a truss, the lateral resistance can be achieved by truss action. As shown in Figure 9, the sloping ramps act as diagonal bracing. A well-defined load path is required to properly transfer
the load to the underlying ground. In this regard, it is essential that the sloping ramps be connected at the lowest extremity of the structure, by proper detailing, to transfer the load to the underlying ground base. Though truss action helps carry lateral forces to the ground, care should be taken so that it does not cause an unanticipated volume change restraint.

In low and moderate seismic zones, the ramp truss is used in resisting lateral loads. However, in high seismic areas where structural ductility is a prime consideration, the contribution of ramp trusses toward the structural stiffness is completely ignored and a lateral load resisting system, separate and apart from the ramp truss is provided. However, the structural stiffness of the ramp truss in high seismic zone is of concern. It has been suggested that, in parking facilities located in high seismic regions where ductile moment frames are used as the primary lateral load resisting system, the sloping ramp be isolated at every floor so that the ramp does not act as a brace.15

POST-TENSIONED MOMENT FRAMES

In post-tensioned parking facilities, post-tensioned beams and CIP reinforced column comprise a moment frame, commonly termed Post-tensioned moment frame. Generally, the post-tensioned beams used in parking facilities span 55 feet or more and the floor-to-floor column height is 10 feet minimum. The post-tensioned frames are designed to carry gravity load. However, the frames can, in addition, resist wind and seismic loads in low and moderate zones without any premium. Because of this inherent strength and stiffness, the post-tensioned construction offers economy not available in precast construction.

Frames Part of Lateral Load Resisting System

The building codes categorize the post-tensioned frames with customary detailing as ‘ordinary’ or ‘intermediate’ frames and allow the frames to comprise, or to be a part of, the lateral load system in low to moderate seismic regions. However, the codes have insisted on the use of non-prestressed ductile steel as the sole reinforcement disallowing the use of prestressing steel in moment frames resisting seismic forces in high seismic zones. Such frames are called ‘special’ or ductile’ frames. In light of the test studies on behavior of prestressed beams and satisfactory performance of post-tensioned systems in earthquakes, the building code have permitted the some use of high strength unbonded tendons in frames that are part of lateral load resisting system in high seismic regions. In allowing the use of prestressing in ductile frames, the IBC code has prescribed that following limiting conditions should be met:

1. The post-tensioning stress is limited to 700 psi (5 MPa) or 1/6 of the beam concrete strength calculated for the beam's rectangular dimensions.
2. The post-tensioning tendons shall not provide more than one quarter of the strength of both positive and negative moments at the beam-column joint.
3. The tendon anchorages are located outside the beam-column joint.
To satisfy the second condition noted above, considerable quantity of mild steel bottom reinforcement is needed at the beam-column joint. This is a highly uneconomical proposition for a post-tensioned frame. Therefore, very few parking structures have been constructed using the code provision. Additional studies are needed to evaluate the code requirements for post-tensioned frame design.

Frames not Part of Lateral Load Resisting System

In addition to being used as part of the lateral load-resisting systems, the post-tensioned concrete frames are also used as frames not part of a lateral load resisting system. This is the case when the frame as a whole or a part of it does not meet the code’s detailing requirements. The codes require that all members not part of the lateral force resisting system must be capable of resisting actions induced by the inelastic distortion of the structure in addition to actions caused by gravity loads. This bifurcation occurs only in the high seismic regions. In such instances, the frame’s stiffness is ignored and the frame is considered to “go far a ride” with the lateral load resisting system. When the shearwalls are used as the lateral load resisting system, the imposed displacement on the non-seismic post-tensioned frames is small. However, if the ductile frames are used to resist the seismic loads, the imposed displacements on frames not part of the lateral load resisting is harder to achieve without significant cracking of the frame columns. In order for the frames to have the prescribed displacement with cracking isolated in pre-assigned regions, hinges are introduced to soften-up the post-tensioned frame. A hinging detail is shown in Ref. 10.

Beam-column Joints

The behavior of beam-column joints in post-tensioned concrete frames during strong ground motions has been a concern. During the Alaska earthquake of 1964, certain beam-column joints failed. The failure was not caused by the post-tensioning, but by the misplacement and inadequate bursting force reinforcement. The detrimental effects of the eccentricity of tendons on cyclic capability of beam-column joints has been a major concern. The eccentricity can be eliminated if the center of gravity of tendons in the joint is lowered and made co-centric with the center of gravity of the concrete beam section. Such a design reduces the amount of joint shear reinforcement as the post-tensioning serves to maintain the joint integrity. However, it is perceived to be uneconomical and is rarely used. Guidelines are needed to educate practicing engineers on design of post-tensioned beam-column joints in high seismic regions.

Abruptness of the failure of couplers and anchorages is another concern in using post-tensioned concrete as part of the lateral load-resisting system under strong ground motions. To address the concern, the ACI Specifications for unbonded single-strand tendons, anchorages and couplers require static tests with minimum standards for strength and static ductility. In addition, tendons are required to withstand a 500,000-cycle low stress test to prove that the tendons have the capability to resist cyclic loading resulting from the expected service loads and building vibrations expected to occur over the useful
life of a commercial building. Further, tendon assemblies are required to withstand a 50-cyle high stress test to simulate the effects of a severe earthquake on the assemblies.

For the beam-column joints which house anchorages, tendon slip may cause a reduction in energy dissipation by pinching of the hysteresis loops. However, the risk of such slippage is very low if the anchorage assemblies meet the fatigue testing and quality requirements of Ref. 5. For post-tensioned concrete frames that are a part of lateral load-resisting system in high seismic regions, IBC requires that the anchorages be placed outside the beam-column joints. This is an excellent measure to minimize or eliminate the risk of anchorage failure.

CONCLUSIONS

The building codes have incorporated the lessons learned from the earthquakes into the design and, as a result, the post-tensioned concrete parking facilities have performed very well in recent earthquakes. The building codes have allowed the use of partial post-tensioning in special moment concrete frames in high seismic regions. This is a positive step to utilize the inherent strength, stiffness and ductility of post-tensioned frames. However, the design profession has not used the recent provisions due to lack of economic incentive, suggesting that additional work is needed to arrive at workable code provisions.

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REFERENCES


ACI Committee 318, “Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05),” American Concrete Institute, Farmington Hills, Mich., 2005, 440 pp.


Figure 1 – Los Angeles International Airport Terminal 6 Parking Facility, Los Angeles, California. (Prime Consultant: Walker Parking Consultants; Architect: Lee & Sakahara.)

Figure 2 – Anaheim Amusement Park Parking facility, Anaheim California. (Prime Consultant: Walker Parking Consultants; Architect: Wolf Architecture).
Figure 3 – Collapsed Diaphragm - CSU at Northridge Parking Structure.  
(Photo courtesy of S. K. Ghosh Associates)
Figure 4 – Beam-column connection in imminent collapse state with collapsed column support – CSU Northridge Parking Structure. (Photo courtesy of S. K. Ghosh Associates)
Figure 5 – Close-up view of collapsed Lateral Load Resisting Frame – CSU at Northridge Parking Facility. (Photos courtesy of S. K. Ghosh Associates)

Figure 6 – Collapsed columns at a post-tensioned parking facility during Northridge Earthquake. (Photo courtesy of S. K. Ghosh Associates)
Figure 7 – Close-up view of a collapsed column during Northridge earthquake.

Figure 8 – SEATAC International Airport Parking Facility with stand-alone helices.
Figure 9 – Isometric view of a double threaded Parking structure.

Figure 10 – Post-tensioned slab-beam-girder framing. SEATAC International Airport Parking Facility, Seattle, Wash.
Figure 11 – Temporary hinge at concrete column base to facilitate precompression in post-tensioned floor system.
Figure 12 – Ramp truss action in parking facilities.
Overview of ACI 440.4R-04 Document on Prestressing Concrete Structures with FRP Tendons

by R. El-Hacha, T.I. Campbell and C.W. Dolan

Synopsis: This paper provides an overview of the ACI 440.4R-04 document on “Prestressing Concrete with FRP Tendons” reported by ACI Committee 440 on “Fiber-Reinforced Polymer Reinforcements”. The document is one of the Emerging Technology Series published by ACI. The paper outlines the content of the document and the philosophy of applying FRP technology as opposed to conventional steel for prestressing. The document offers general information on the history and use of FRP for prestressing applications, and a description of the unique material properties of FRP. It also focuses on the current state of design, development, and research needed to characterize and ensure the performance of FRP as prestressing tendons in concrete structures. The proposed guidelines are based on knowledge gained from worldwide experimental research, analytical work, and field applications of FRPs used as prestressed tendon. Current developments in the document include a basic understanding of flexural and axial prestressed members with FRP tendons, FRP shear reinforcement, bond of FRP tendons, and unbonded or external FRP tendons for prestressing applications. The document concludes with a list of research needs.

Keywords: anchorage; bond length; crack; deflection; deformation; development length; ductility; fatigue; flexure, jacking stresses; post-tensioning; pretensioning; prestressed concrete; reinforcement ratio; shear; tendon
BIOGRAPHY

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INTRODUCTION

Fiber-reinforced polymer (FRP) composites have been proposed for use as prestressing tendons in concrete structures. The promise of FRP materials lies in their high-strength, lightweight, non-corrosive, non-conducting, and non-magnetic properties. At present, the higher cost of FRP materials suggests that FRP use will be confined to applications where the unique characteristics of the material are most appropriate, such as in a highly corrosive environment. Efficiencies in construction and reduction in fabrication costs will expand their potential market.

FRP reinforcement is available in the form of bars, grids, plates, and tendons. One of the principal advantages of FRP tendons for prestressing is the ability to configure the reinforcement to meet specific performance and design objectives. FRP tendons may be shaped as rods, bars, or strands, and are typically made from one of three basic fibers: aramid, carbon, and glass. Additionally, FRP composites are heterogeneous and anisotropic. The final characteristics of an FRP tendon are dependent on fiber and resin properties, as well as on the manufacturing process. Specific details of a particular tendon should be obtained from the manufacturer of the tendon.

A FRP prestressing system consists of the tendon and the anchorage. Properties, performance, and overall behavior are dependent on the tendon/anchorage system and on the individual components. Performance of independent elements should be verified by tests.
During the late 1980s and throughout the 1990s, several demonstration projects have shown the potential of FRP prestressing in concrete bridges. In 1993, a two spans bridge was built in Calgary, Alberta, using pretensioned concrete girders containing FRP tendons incorporating fiber optic sensors (Rizkalla and Tadros 1994). This was the first bridge of its kind in Canada, and one of the first in the world. A second bridge incorporating FRP prestressing tendons was built at Headingly, Manitoba, in 1997. Lawrence Technological University has also been involved with a demonstration bridge using external unbonded FRP tendons (Grace 1999). The bridge in Southfield, Michigan, was completed in 2001 and is a three span structure that contains bonded and unbonded Carbon Fiber Reinforced Polymer (CFRP) tendons in both longitudinal and transverse directions (Grace et al. 2002(a) and (b)). The worldwide number of prestressed FRP applications is less than 100 (MDA 2004 and IABSE 2003). Most are bridge structures where issues of fire were not considered critical.

In 1993, the first design guidelines for FRP-reinforced and prestressed concrete buildings were established by the Japanese Society of Civil Engineers. The Japanese version of the guideline was released in 1995, while the English version (Sonobe et al. 1997) was published in 1997. Work relating to FRP prestressing in the United States has been documented by Dolan (1999). In 1994, the Federal Highway Administration (FHWA) sponsored research into development of design recommendations for FRP prestressing for bridge girders that led to design specification recommendations for the American Association of State Highway and Transportation Officials (AASHTO) (Dolan et al. 2000).

In Canada, several researchers, mostly within ISIS Canada (Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures), have been investigating applications of FRP prestressing tendons. The Canadian Standards Association has produced two standards CAN/CSA S6-00 (CSA 2000) and CAN/CSA S806-02 (CSA 2002) which contain provisions for the use of FRP prestressing tendons in bridges and buildings, respectively.

ORGANIZATION OF DOCUMENT

The ACI 440.4R-04 document is comprised of ten chapters. Chapter 1 presents an introduction, the historical development and use of FRP reinforcement, research efforts, and demonstrations and field applications. Chapter 2 is devoted to FRP tendon and anchorage characterizations, and descriptions of available commercially FRP tendons. Chapter 3 illustrates the flexural design and capacity prediction of concrete members prestressed with FRP tendons, including strength design methodology, allowable flexural service and jacking stresses, loss of prestress, and ductility requirements. Chapter 4 is dedicated to serviceability requirements including deflection limitations, crack width and spacing, and fatigue. Shear strength, together with spacing limitations, minimum amount and detailing of shear reinforcement, is presented in Chapter 5. Chapter 6 includes the determination of transfer and development lengths of various types of FRP tendons. Post-tensioning with unbonded or externally prestressed FRP tendons is presented in Chapter 7. Demonstration studies of concrete piles prestressed with FRP tendons are
presented in Chapter 8. Finally, Chapter 9 is devoted to the major research needs that remain to be investigated. A list of referenced standards and reports and cited references are given in Chapter 10. A design example on a simply supported pretensioned T-beam is presented in Appendix A using dual units (SI and Imperial).

LIMITATIONS OF THE DOCUMENT

The emphasis of the ACI 440.4R-04 document is on flexural members in concrete buildings and bridges pretensioned with aramid or carbon FRP tendons. The document examines both internal and external FRP prestressed reinforcement in the form of tendons. Information is provided for bonded and unbonded post-tensioned applications where it is available. Only fully prestressed members are considered with no attempt being made to address partially prestressed members. The committee feels that this document is relevant to simple spans and to spans made continuous by placing steel reinforcement in the deck of a bridge structure. No recommendations are made for beams made continuous with FRP tendons or for moment resisting frames where ductility or large deformations are required for resistance to seismic loading.

FRP TENDONS

A FRP tendon is identified by the type of fiber used in making the tendon. These fibers are aramid, carbon, and glass. The selection of the fiber is primarily based on consideration of cost, strength, stiffness, and long-term stability. Only aramid and carbon FRP tendons are recommended in the ACI 440.4R-04 document for prestressing applications. Glass fibers have poor resistance to creep under sustained loads and are more susceptible to alkaline degradation than carbon and aramid fibers. The resins used for fiber impregnation are usually thermosetting and may be polyester, vinylester, epoxy, phenolic or polyurethane. Final characteristics of a FRP tendon are dependent on fiber and resin properties, as well as manufacturing process. The properties and characteristics of commercially available FRP tendons, based on manufacturer’s published data, are presented in the document and some are summarized in Table 1 of this paper. The various FRP tendons described are Arapree®, FiBRA, Technora®, Parafil®, Leadline™, and CFCC. Some of these tendons are shown in Figure 1 of this paper.

Aramid FRP

The term aramid comes from its chemical bases as ARomatic polyAMIDe. Aramid fibres have lower weight and a lower Young’s modulus than carbon fibres. Aramid fibres are superior to carbon fibres in terms of toughness and impact resistance. Commercially they are available in four main types: (1) Arapree® (Italy) in which two types of cross section are available in the marketplace: rectangular and circular, (2) FiBRA (Japan) in which, depending on the manufacturing process, two types of rod are available, rigid and flexible, (3) Technora® (Japan) is a spirally wound pultruded tendon impregnated with a vinyl ester resin, and (4) Parafil® (UK) is a parallel-lay tendon composed of dry fibers contained within a protective polymeric sheath.
Carbon FRP

Carbon fibers provide advantages that include high-strength and high stiffness-to-weight ratios, excellent fatigue properties, excellent moisture resistance, high temperature and chemical resistance, and electrical and thermal conductivity. Due to low ultimate strains, carbon fibers possess a low impact resistance. Carbon FRP is available in the form of bars, multi-wire strands, ropes, and cables. Commercial CFRP tendons are available in two different types as (1) Leadline™ CFRP (Japan) that is pultruded and epoxy impregnated using Dialead coal tar pitch-based continuous carbon fibers and an epoxy resin. Leadline CFRP™ tendons are formed with a smooth or an indented surface, and (2) Carbon Fiber Composite Cables (CFCC) that is made using synthetic fibers known as Polyacrylonitrile (PAN). CFCC tendons are formed by twisting a number of rods in a manner similar to a conventional stranded steel tendon. Both Leadline™ and CFCC tendons are produced in Japan.

ANCHORAGE SYSTEMS FOR FRP TENDONS

FRP tendons are heterogeneous and transversely isotropic, being rather weak with respect to the transverse axis and comparatively strong with respect to the longitudinal axis (direction of fibers). FRP tendons are sensitive to transverse pressure when subjected to high axial stress. High transverse pressure and surface notching reduce the effective tensile strength. The very high ratio of axial to lateral strength of the FRPs (as high as 30:1) translates into a need to rethink and redesign the anchoring system for FRP tendons. Conventional anchorage systems used for steel strands cannot be used for FRP tendons since premature anchorage zone failure could occur.

Different types of anchorages developed for the various types of FRP tendons are described in the ACI 440.4R-04 document under the following categories: clamp, plug and cone, straight sleeve, contoured sleeve, metal overlay, and split-wedge anchorages. Some of these anchorages are shown in Figure 2. The anchorage developed for Arapree®, both flat and round rod types, consists of a tapered metal sleeve into which the tendon is either grouted (post-tensioning application) or clamped between two wedges. FiBRA has two different types of anchorage: a resin-potted anchorage used for single-tendon anchoring, and a wedge anchorage for either single- or multiple-tendon anchoring. Technora® tendons employ either wedge-type or potted-type anchorages. Parafil® tendons are anchored by means of a barrel and spike fitting, which grips the fibers between a central tapered spike and an external matching barrel. Leadline™ employs a wedge system to anchor the tendons with an aluminum sleeve that fits between the wedges and the tendon. The anchorages for CFCC tendons are chosen based on the intended application as resin-filling and die-cast methods. The resin-filling method bonds the cable to a steel cylinder utilizing a high-performance epoxy. These cylinders can be threaded as necessary to allow anchoring with nuts. The die cast method attaches a die-molded alloy and steel tube to the cable. Steel wedges then clamp the steel tube in a similar manner to steel tendon systems.

The various failure modes of anchorages are summarized into two main categories: failure of the anchorage system and rupture of the FRP tendons outside the anchorage. It
is recommended that, for a satisfactory anchorage design, the FRP tendon should not break in or within three tendon diameters of the anchorage. Failure of the anchorage system has been classified into four modes: (i) movement or slip of the tendon out of the anchorage caused by insufficient grip between the tendon and sleeve, (ii) slip of the sleeve and tendon together relative to the wedges, (iii) slip of the wedges relative to barrel, and (iv) rupture of the tendon inside the anchorage.

**FLEXURAL DESIGN OF BONDED PRESTRESSED MEMBERS**

A conventional prestressed concrete beam with steel tendons will deform elastically until cracking, and then the rate of member deflection will progressively increase as the tendons yield until failure occurs either by concrete crushing or tendon rupture. On the other hand, an FRP prestressed beam will deform elastically until cracking, then continue to deform in an approximate linear manner under increasing load until the tendon ruptures or the concrete crushes. These two types of behavior are compared in Figure 3. The lower modulus of elasticity of FRP is reflected in the lower post-cracking behavior.

**Strength Design Methodology**

The approach to strength design of FRP prestressed beams is based on the concept of a balanced reinforcement ratio, \( \rho_b \), which is defined as the reinforcement ratio that results in simultaneous rupture of the FRP tendons and crushing of the concrete. Concrete failure is taken as an extreme compression strain of \( \varepsilon_{cu} = 0.003 \). A rectangular stress block is used to model the concrete behavior. Tendon failure is defined as occurring when the strain in the tendon reaches the ultimate tensile strain capacity.

**FRP Balanced Ratio**

Strain compatibility and equilibrium of internal forces on a cross section shown in Figure 4 allow determination of the balanced reinforcement ratio in terms of material properties:

\[
\rho_b = 0.85 \beta_1 \frac{f_c'}{f_{pu}} \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{pu} - \varepsilon_{pe} - \varepsilon_{d} - \varepsilon_{pr}}
\]

where

\( f_c' \): specified compressive strength of concrete, MPa (psi)

\( f_{pu} \): design ultimate tensile strength of prestressed FRP tendon and anchorage system, MPa (psi)

\( \beta_1 \): stress-block factor for concrete

\( \varepsilon_{cu} \): ultimate strain in concrete in compression

\( \varepsilon_{pu} \): ultimate tensile strain in the prestressed FRP tendons

\( \varepsilon_{pe} \): effective strain in the FRP tendon after all losses
\[ \varepsilon_d : \text{additional strain in tendon that causes the extreme precompressed fiber to reach zero strain (decompression)} \]

\[ \varepsilon_{pr} : \text{loss of strain capacity due to sustained loads} \]

The strain loss due to sustained loads, \( \varepsilon_{pr} \), is nearly zero if the strain in the tendons due to sustained load is less than 50% of the ultimate tensile strain, and is recovered at the nominal strength condition. The decompression strain, \( \varepsilon_d \), is typically an order of magnitude less than the flexural strain. Thus, setting these two strain values to zero gives the following simplified definition for \( \rho_b \).

\[
\rho_b = 0.85\beta_1 \frac{f_c'}{f_{pu}} \left( \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{pu} - \varepsilon_{pe}} \right)
\]

**Flexural Design and Capacity Prediction**

The design for both bonded and unbonded construction is explained in the document. The flexural behavior of a beam prestressed with bonded FRP tendons is described according to whether the critical section is either a compression- or a tension-controlled section. A compression-controlled section condition occurs when the reinforcement ratio \( \rho \) is greater than \( \rho_b \), and the concrete crushes prior to failure of the tendons. When \( \rho \) is less than \( \rho_b \), a tension-controlled section condition occurs as a result of rupture of the tendons before crushing of the concrete.

**Strength Reduction Factors for Flexure**

Similar to the ACI 318-02, strength reduction factors are used in which the variation in strength reduction factors is based on net tensile strain in the reinforcement. A transition zone between tension-controlled and compression-controlled is also presented. Tension-controlled sections have a net tensile strain, strain in the reinforcement farthest from the compression face, greater than or equal to 0.005, while compression controlled-sections have a net tensile strain less than or equal to 0.002. The transition in strength reduction factor is indicated in Figure 5.

**Flexural Capacity for Vertically Distributed Tendons**

A method for determining the flexural strength capacity of a section with vertically aligned tendons (i.e. the prestressing tendons are not all located at the same depth from the compression face), based on the stress and the strain distribution throughout the depth of the section, is presented in the document. As a possible means of optimizing the design of a section the level of prestress may be varied in the tendons at each depth so that the final stress distribution in the tendons is uniform with depth at maximum curvature.
The concrete service load stresses adopted in the document are the same as those specified by the AASHTO Standard Specification for Highway Bridges (1998), which are the same as in ACI 318-02, except that the limitations on the tensile stresses in the concrete are more restrictive.

Usually steel tendons are typically stressed to 82% of their yield stress or approximately 0.005 strain. However, the allowable jacking stresses in FRP tendons adopted in the document are typically limited to 50 to 65% of their ultimate strength due to stress-rupture limitations for AFRP and CFRP tendons, respectively. This lower range of allowable stress actually corresponds to strains between 0.008 and 0.012, or 1.5 to 2.5 times the typical prestressing strain used in steel tendons. The allowable stress immediately following transfer is limited to 40 to 60 % of their ultimate strength for AFRP and CFRP tendons, respectively.

Correction of Stress for Harped Tendons

Since FRP tendons are linear elastic to failure, a correction for the stress level in draped or harped FRP tendons must be considered. This will effectively result in a loss of tendon strength due to the increased strain induced by the curvature resulting from the drape or harping of the FRP tendons. Dolan et al. (2000) proposed a stress increase due to harping as follows:

\[ f_h = \frac{E_f R_t}{R} \]

and a combined stress in a tendon at a harping saddle due to the jacking load, given by the following equation, and less than the allowable stress values:

\[ f = \frac{P_j}{A_p} + \frac{E_f R_t}{R} \]

where

- \( E_f \): modulus of elasticity of FRP tendon, MPa (psi)
- \( R_t \): radius of the FRP tendon, mm (in)
- \( R \): radius of curvature of the harping saddle, mm (in)
- \( P_j \): jacking load, N (lb)
- \( A_p \): cross-sectional area of FRP tendon, mm\(^2\) (in\(^2\))

Prestress Losses

The document describes the various types of loss of prestress in FRP tendons. Anchorage seating loss at transfer of prestress is a function of the tendon system. Traditional losses due to creep, shrinkage, and elastic shortening of concrete may be calculated using the same standard methods as for concrete sections prestressed with steel tendons (PCI 1975, 2000). Relevant friction and wobble coefficients should be obtained from the
manufacturer of the prestressing system used. Relaxation losses in the FRP tendons results from three sources: (i) relaxation of polymer; (ii) straightening of fibers; and (iii) relaxation of fibers. Estimation of the total relaxation loss expressed as a percentage of the stress at transfer is presented in the document by assessing these three effects separately. In general loss of prestress will be lower in FRP tendons than in steel tendons due to lower values of the modulus of elasticity and lower relaxation.

DUCTILITY OR DEFORMABILITY

Because FRP tendons are brittle materials and do not exhibit ductility under the traditional definition, care should be taken to ensure that sufficient warning is exhibited before failure. Due to the lack of ductility, the concept of deformability as an index to measure performance provides a method of ensuring that this warning exists (Dolan et al. 2000). Deformability may be defined as the ratio of deflection at ultimate to deflection at cracking. Using the latter definition, FRP prestressed members can have considerable deformability. Another approach to define the deformability is through the ratio of curvatures under ultimate loads to those same quantities under service loads as given by:

\[
DI = \frac{(1-k)\varepsilon_{pu}}{\left(1-\frac{a}{d\beta_i}\right)\varepsilon_{ps}}
\]

where

\[
a = \frac{\rho df_{pu}}{0.85 f'c}\]

\[
k = \text{ratio of neutral axis depth to FRP tendon depth}
\]

\[
a = \text{depth of the equivalent compression block, mm (in.)}
\]

\[
d = \text{depth to the FRP tendon, mm (in.)}
\]

SERVICEABILITY

Determination of the short- and long-term deflections for FRP prestressed beams are outlined in the document. For the short-term deflection, an equation to calculate a modified effective moment of inertia after cracking is proposed that takes into account the softening effect that cracking has on concrete members prestressed with FRP tendons.

\[
I_e = \left(\frac{M_{cr}}{M_a}\right)^3 \beta_d I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \leq I_g
\]

where

\[
\beta_d: \text{factor to soften the effective moment of inertia: } \beta_d = 0.5 \left(\frac{E_f}{E_s} + 1\right)
\]

\[
E_f: \text{modulus of elasticity of FRP tendon, MPa (psi)}
\]
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$E_s$: modulus of elasticity of steel, MPa (psi)

$I_g$: gross moment of inertia, mm$^4$ (in$^4$)

$I_{cr}$: cracked moment of inertia, mm$^4$ (in$^4$)

$M_{cr}$: cracking moment, N.mm (in.-lb)

$M_a$: maximum moment at which the deflection is being computed, N.mm (in.-lb)

For a rectangular section or a flanged section with $kd < h_f$, (where $h_f$ is the depth of the flange) the cracked moment of inertia can be calculated from:

$$I_{cr} = \frac{b(kd)^3}{3} + nA_p(d - kd)^2$$

It is proposed that the long-term deflection may be calculated in a manner similar to that for conventional steel prestressed concrete members (PCI, 2000) by introducing multipliers to adjust the deflection for FRP prestressed concrete member.

It is unlikely that fatigue will be a problem in uncracked members as the stress range in the tendons under repeated loading will be small (Grace 2000).

SHEAR STRENGTH OF FRP-PRESTRESSED MEMBERS

Members prestressed with FRP tendons behave similarly to members prestressed with steel tendons. An approach that conservatively modifies the ACI 318-02 expressions to account for the special characteristics of FRP stirrups has been suggested in the document to determine the nominal shear strength of concrete members prestressed with FRP tendons. The nominal shear strength is considered as the sum of the shear resistance provided by concrete, $V_c$, the shear resistance provided by the FRP stirrups, $V_{frp}$, that takes into account the strength reduction, $\phi_{bend}$, due to bending of the stirrup, and the shear resistance provided by the vertical component of prestressing force, $V_p$:

$$V_n = V_c + V_{frp} + V_p$$

where

$$V_c = 0.17\sqrt{f'c}b_wd \quad (N) \quad (V_c = 2.0\sqrt{f'c}b_wd \quad (lb))$$

$$V_{frp} = \frac{f_{fb}A_vd}{s}$$

$f_{fb}$: stress in the bent FRP stirrup, MPa (psi), $f_{fb} = \phi_{bend}f_{fu}$

$$\phi_{bend} = \left(0.11 + 0.05\frac{r}{d_b}\right) \quad \text{and} \quad 0.25 \leq \phi_{bend} \leq 1.0$$
Limits on the maximum spacing for FRP stirrups have been proposed in the document to be the same as recommended by ACI 318-02 for non-prestressed members. To prevent shear failure in members where the sudden formation of shear cracks can lead to excessive distress, minimum recommended amount of FRP shear reinforcement is suggested in the document. The proposed expression is a modification to the ACI 318-02 expression for steel stirrups but using the effective strength of the FRP stirrup at the bends instead of the yield strength of reinforcing steel stirrup to conservatively estimate the minimum amount of FRP stirrups. Because of the different mechanical properties of FRP and steel stirrups, the document also presents detailing of the FRP shear stirrups in which a minimum ratio of the bend radius to the bar diameter, \( r/d_b \), of 3 and a minimum tail length of 12\( d_b \) are recommended. In addition, FRP stirrups should be closed stirrups with 90-degree hooks.

**BOND AND DEVELOPMENT**

The transfer and development length of a FRP tendon is a function of the configuration of the perimeter and the surface condition of the FRP, the stress in the FRP, the method used to transfer the prestressing force to the concrete, and the tensile strength and cover of the concrete. The mechanism of bond to concrete differs between FRP and steel tendons due to differences in shapes, surface treatments, and elastic moduli. The surface texture of FRP tendons may vary, resulting in bond with the surrounding concrete that varies from one tendon configuration to another. The transfer and development lengths for various types of FRP tendons (Arapree®, FiBRA, Technora®, Leadline™, and CFCC) as function of the FRP tendon diameter are discussed in the document. Equations to determine the transfer length and the development length of CFRP tendons are suggested. Typical values for transfer and development lengths of various FRP tendons are suggested in the document and summarized in Table 2.

**UNBONDED AND EXTERNAL TENDON SYSTEMS**

For an unbonded prestressed member where there is no strain direct compatibility between the concrete and the tendons, the stress in the tendons at ultimate for an under-reinforced beam may be determined using the approach suggested by Naaman et al. (2002). This approach uses a strain-reduction coefficient to relate the strain in an unbonded tendon to that in an equivalent bonded tendon. Knowing the strain in the tendons, the moment capacity may be computed as for a bonded section. The strain reduction coefficient at ultimate depends on a number of variables, such as loading configuration and extent of the cracks in a beam, and varies theoretically between 0 and
The stress in the prestressing tendons at failure of the beam is given by:

\[ f_p = f_{pe} + \Omega_u E_p \varepsilon_{cu} \left( \frac{d_p}{c_u} - 1 \right) \]

where

- \( f_{pe} \): the effective prestress in the tendon when the beam carries only the dead load after the prestress losses have occurred, MPa (psi)
- \( E_p \): modulus of elasticity of the prestressed tendon, MPa (psi)
- \( \varepsilon_{cu} \): the strain in the extreme compression fiber at ultimate
- \( d_p \): depth from the concrete top fiber to the centroid of the prestressing tendons, mm (in)
- \( c_u \): depth of the neutral axis at ultimate, mm (in)
- \( L \): length of the prestressing tendon between anchorages, mm (in.)
- \( \Omega_u \): the strain reduction coefficient at ultimate, given by:

\[ \Omega_u = \begin{cases} \frac{1.5}{(L/d_d)} & \text{(for one-point loading)} \\ \frac{3.0}{(L/d_d)} & \text{(for two-point or uniform loading)} \end{cases} \]

One of the main differences in design between external and internal unbonded tendons is the variation of eccentricity in the case of external tendons during deformation of the beam under load. In addition, the effect of bending at the deviator points on the ultimate strength of the tendons should be taken into consideration when using FRP as externally prestressed harped tendons. The concept of depth reduction factor, \( R_d \), to estimate the effective depth, \( d_e \), of an external tendon at ultimate has been introduced:

\[ d_e = R_d d_p \]

where:

\[ R_d = 1.14 - 0.005 \left( \frac{L}{d_d} \right) - 0.19 \left( \frac{S_d}{L} \right) \leq 1.0 \quad \text{(for one-point loading)} \]

\[ R_d = 1.25 - 0.010 \left( \frac{L}{d_d} \right) - 0.38 \left( \frac{S_d}{L} \right) \leq 1.0 \quad \text{(for third-point loading)} \]

\( S_d \): spacing of the deviators and \( L \) is the span of the beam, mm (in)
Different expressions for the strain reduction coefficient at ultimate for external FRP tendons are suggested in the document.

\[
\Omega_u = \frac{0.21}{(L/d_d)} + 0.04 \left( \frac{A_{p\text{ int}}}{A_{p\text{ tot}}} \right) + 0.04
\]  
(for one-point loading)

\[
\Omega_u = \frac{2.31}{(L/d_d)} + 0.21 \left( \frac{A_{p\text{ int}}}{A_{p\text{ tot}}} \right) + 0.06
\]  
(for third-point loading)

where

\[ A_{p\text{ int}} \]: area of the internal prestressed reinforcement, mm\(^2\) (in\(^2\))

\[ A_{p\text{ tot}} \]: total area of internal and external prestressed reinforcement, mm\(^2\) (in\(^2\))

External tendons are designed to ensure longitudinal prestressing of a beam and generally represent only a portion of the total flexural reinforcement. The remaining reinforcement may consist of internal prestressed or non-prestressed reinforcement, or a combination of both, depending on the structural system and the type of construction. A minimum amount of bonded reinforcement is also necessary to control the distribution of cracks and to limit the crack widths.

**PILE DRIVING AND IN-PLACE FLEXURE**

The document describes the different types of damage that may occur during driving of prestressed concrete piles, in addition to the various demonstration studies conducted in the United States in which full-size carbon FRP prestressed concrete piles were driven and monitored to verify their performance. Guidelines for FRP prestressed piles are presented in the document including the minimum specified strength of concrete, range of the effective prestress, the FRP spiral ties capacity, and the monitoring of the driving stresses.

**RESEARCH NEEDS**

A number of research areas identified by the committee as major concerns that remain to be investigated are presented in the document. Such areas include:

- the development of economical FRP tendon and anchorage systems;
- evaluation of fatigue effects at anchorages for unbonded systems and fire protection of anchorages;
- development of harping and saddle devices that would reduce the stress concentration at the harping points of the FRP tendons;
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- studies on the long-term durability behavior of bond in FRP prestressing tendons;
- investigation galvanic action;
- use of external FRP post-tensioning for rehabilitation.

REFERENCES

The document includes a comprehensive list of referenced standards and reports, and cited references. The following references have been used in this paper:


ACI Committee 440, 2004, “Prestressing Concrete with FRP Tendons (ACI 440.4R-04),” American Concrete Institute, Farmington Hills, Mich., 35 pp.

ACI Committee 318, 2002, “Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (318R-02),” American Concrete Institute, Farmington Hills, Mich., 443 pp.


ACKNOWLEDGMENTS

The authors are members of the ACI Committee 440 on “Fiber-Reinforced Polymer Reinforcements”. Drs. Campbell and Dolan were the past Co-Chairs of the ACI Subcommittee 440-I on “FRP Prestressed Concrete” and Dr. El-Hacha is the current Co-Chair. The authors wish to acknowledge the direct and indirect contributions of the members of ACI Committee 440 and all individuals involved in the preparation of its documents.
Table 1 – Typical mechanical properties of FRP tendons (compiled from various references)

<table>
<thead>
<tr>
<th>Property</th>
<th>AFRP</th>
<th>CFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fiber volume ratio</td>
<td>Arapree® Twaron</td>
<td>Technora® Kevlar49</td>
</tr>
<tr>
<td>Density (g/cm³)</td>
<td>0.45</td>
<td>0.65</td>
</tr>
<tr>
<td>Longitudinal tensile strength (GPa)</td>
<td>1.2-1.5</td>
<td>1.25-1.4</td>
</tr>
<tr>
<td>Transverse Tensile strength (MPa)</td>
<td>57</td>
<td>-</td>
</tr>
<tr>
<td>Longitudinal Modulus (GPa)</td>
<td>62-64</td>
<td>65-70</td>
</tr>
<tr>
<td>Transverse Modulus (GPa)</td>
<td>5.5</td>
<td>-</td>
</tr>
<tr>
<td>Major Poisson’s ratio</td>
<td>0.38</td>
<td>0.34-0.6</td>
</tr>
<tr>
<td>Bond strength (MPa)</td>
<td>7.7</td>
<td>10-13</td>
</tr>
<tr>
<td>Maximum longitudinal strain (%)</td>
<td>2.4</td>
<td>2.0-3.7</td>
</tr>
<tr>
<td>Longitudinal thermal expansion coefficient/°C</td>
<td>-2x10⁻⁶</td>
<td>-2x10⁻⁶</td>
</tr>
</tbody>
</table>

A “—” indicates that information is not available or is not applicable.

1 MPa = 145 psi; 1GPa = 145 ksi; 1 g/cm³ = 62.4 lb/ft³; 1/°C = 1.8/°F

Table 2 – Typical transfer and development lengths for FRP tendons

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>Diameter mm</th>
<th>Young’s modulus MPa</th>
<th>Tensile strength MPa</th>
<th>f_p₀/f_pₙ</th>
<th>Lₖ/dₖ</th>
<th>L₉/d₉</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aramid</td>
<td>Arapree®</td>
<td>9.9</td>
<td>127,600</td>
<td>2450</td>
<td>0.5 to 0.7</td>
<td>16 to 50</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>FIBRA</td>
<td>10.4</td>
<td>48,270</td>
<td>1430</td>
<td>0.4 to 0.6</td>
<td>20 to 50</td>
<td>90</td>
</tr>
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Figure 1 - Samples of FRP tendons

Figure 2 - Types of anchors for FRP tendons
Figure 3 - Schematic representation of moment-deflection responses for a concrete element prestressed with either steel strands or FRP tendons.

Figure 4 - Balanced ratio - stress and strain conditions.

Figure 5 - Variation in strength reduction factor with net tensile strain.
Variable Thickness Barrel Anchor for CFRP Prestressing Rods

by A. Al-Mayah, K. Soudki, and A. Plumtree

Synopsis: The successful implementation of carbon fiber reinforced polymer (CFRP) rods in prestressed applications depends on the anchor system. Finding a proper anchor is a challenging problem due to the weakness of the CFRP tendon in the transverse direction. This paper presents a finite element study conducted to investigate the contact pressure distribution in a wedge anchor system for CFRP rods. The effect of the thickness variation of the barrel on the contact pressure distribution was investigated. The thickness of the barrel was reduced at the loading end of the rod. Different thickness reductions were investigated. It was found that as the reduction of the thickness increased the contact pressure decreased at the loading end of the rod. This leads to the elimination of the stress concentration on the carbon fiber rod which results in the avoidance of the premature failure of the rod. Also, higher shear stress was observed on the rod-sleeve surface than on the wedge-barrel surface. Tensile load-displacement relationship was modeled for different barrel thickness reductions. For a given displacement, higher tensile load was carried by the anchor with less barrel thickness reduction.

Keywords: anchor; CFRP; contact pressure; FEM; prestressed concrete
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INTRODUCTION

Fiber reinforced polymer (FRP) rods are becoming increasingly popular as prestressing materials in prestressed concrete for new design, repair and/or strengthening of existing structures. The high strength-to-weight and high stiffness-to-weight ratios, as well as high corrosion resistance make an FRP rod an excellent prestressing material. The successful implementation of FRP rods depends on the anchor systems\(^1\). However, finding a proper anchor is considered a challenging problem for FRP tendon applications\(^2\) due to the weakness of the FRP tendon in the transverse direction. The available experience of steel tendon-anchors cannot be used in developing FRP-rod anchors. An interactive developmental work that includes the mechanical modeling and experimental verification is recommended by Rostásy.\(^7\)

Researchers have developed and investigated different anchors for FRP rods, as reported by Reda Taha and Shrive\(^3\) but there has not been one that has been widely accepted.\(^4\) Depending on the gripping mechanism, these anchor systems include clamps, plug and cone, resin sleeve, potted resin, metal overlay, and split wedges (Fig. 1). However, split wedge anchors have many attractive features to the prestressed concrete industry including compactness, ease of installation and reusability.

Experimental investigations of the FRP rods with wedge anchor systems revealed different failure modes including anchor and rod failures.\(^5\) The anchor failure modes are either slip of the rod, slip of rod-sleeve couple, slip of wedge or rupture of the rod inside the anchor. The slip mode is related to the interfacial shear stress at the specified surfaces. However, the rupture of the rod is a result of high stress concentrations in the rod inside the anchor causing damage of the fibers leading to premature failure. Proper anchor design which provides a low stress concentration and
uniform load distribution in the anchor overcomes this problem and results in failure of the rod outside the anchor after reaching its ultimate strength.

To establish the experience required to design an anchor system for FRP rods, a comprehensive experimental and analytical research program was conducted to investigate the interfacial mechanics of the CFRP rod-metal couple. The results of the interfacial study were used in design different anchor systems based on finite element and mathematical models.

In the area of FRP rod anchor systems, 2D finite element analysis was used to model resin potted anchors used for FRP rods. However these results cannot be applied in wedge anchor systems. Syed-Ahmed et al. developed an axisymmetric finite element model to simulate the differential angle anchor system presented. The sleeve was neglected in the study. Campbell, et al. conducted similar study except the wedges were modeled as orthotropic material. These models gave the radial stress and the longitudinal stress distribution of the anchor along the anchor zone. The complete anchor, including the sleeve, was modeled in 2D to investigate the performance of the anchor using different sleeve materials, coefficients of friction, and presetting of the wedges.

To design an anchor system for FRP rods, three dimensional finite element models were developed in this study to simulate the exact configuration and number of wedges. A variable barrel thickness wedge anchor system was studied. The model was a quarter of the real anchor system including the rod, sleeve, wedge and the barrel. The main purpose of this analysis was to calculate the response of the anchor components inside the anchor. The contact pressure distribution on the rod surface was calculated using different thickness reductions.

**FINITE ELEMENT MODEL**

**General**

Figure 2 shows the geometrical configuration of the variable barrel thickness anchor. The general concept of the variable barrel thickness design was to reduce the contact pressure at the loading end of the rod by reducing the thickness of the barrel by (rd). In addition, to have a smooth transition from contact-free to the anchoring zone at the loading end of the rod, the wedges were allowed to extend out of the barrel. This would leave the wedges free to deform in the outward direction minimizing the stress concentration and avoiding any notching at the loading end of the rod. The thickness of the wedge had to be large enough to carry the bending stress yet small enough to permit bending.

Variable barrel thickness anchors of different barrel thickness reduction (rd) of 0, 7, and 15 mm were modeled in this study (Fig. 3a). Similar sleeve dimensions and materials were used in all anchors (Fig. 3b).
Model Configuration

The anchor configurations have the same boundary conditions and components. The 3D model simulated a quarter of the anchor system using ABAQUS 6.3 finite element package. It consisted of the rod, sleeve, wedges and barrel. An 8-node linear brick element was used for most of the model except for one layer of the rod at its centre where a 6-node triangular element was used. Different meshes were investigated before reaching the final density. The running time for reaching convergence and the size of the memory required were considered in selecting the mesh.

Figure 4 shows the meshing schemes of the anchors. The number of elements in each of the anchor components was distributed as follows: 3 x 72 x 10 for the rod, 1 x 68 x 10 sleeve, 3 x 30 x 10 wedge and 4 x 26 x 10 barrel.

A nonlinear geometry concept was used in the model since a large displacement of different anchor components was either applied or resulted from the movement of another part. By including this important parameter, the elements would be formulated according to the current or instantaneous nodal coordinates.\textsuperscript{13}

Boundary Conditions

The boundary conditions in the model were either constant or variable. The constant boundary conditions were fixed (non-changing) throughout the analysis, whereas the variable boundary conditions simulated the applied load on specific anchor components. There were three constant boundary conditions (Fig. 5): 1) The faces \((a, b, d, e)\) of the rod, \((a, c)\) of the sleeve and \((a, b, c, d, e)\) of the barrel were prevented to move in the \(y\)-direction, 2) whereas \((b, c, f, e)\) of the rod, \((b, d)\) of the sleeve and \((e, f, g, h)\) of the barrel were supported against movement in the \(x\)-direction. 3) While the rod and sleeve were free to move in the longitudinal direction, the barrel was constrained in the longitudinal direction through the supporting face \((a, b, d, e, h)\). The wedge was free to move in any direction.

In the variable boundary conditions, the wedge was inserted inside the barrel by a set of displacements to simulate the presetting process. Subsequently, this boundary condition was removed during the tensile loading. In the tensile loading process, the loaded end of the rod \((a, b, c)\) was pulled by applying a predetermined displacement. This increased by 0.25 mm for each loading step.

Material Properties

The sleeve, barrel and wedges were modeled as isotropic materials. However, the CFRP rod was modeled as an orthotropic material. The material properties of the anchor components are listed in Table 1.
Contact Surfaces and Friction

The model consisted of three contact surfaces namely; wedge-barrel, sleeve-wedge and rod-sleeve. Based on experimental observation, each surface has a specific coefficient of friction. Experimental findings of anchors of similar nature showed that the rod, sleeve and wedges moved together and slip occurred only on the wedge-barrel surface. Therefore, higher grip was expected at the sleeve-wedges and rod-sleeve surface. As a result of the application of lubricant on the wedge-barrel surface, a low coefficient of friction ranging from 0.0 to 0.02 was applied. A high coefficient of friction of 0.4 was applied to simulate the strong grip between the steel wedges and copper sleeve that was also noticed during a pull-out and anchor tests. Due to the nature of the materials that form the rod-sleeve surface, a comprehensive experimental study was conducted to find the coefficient of friction. For the given materials, the coefficient of friction at this surface was 0.24.

RESULTS AND DISCUSSION

Contact Pressure Distribution

For a presetting distance, the distance in which the wedges were pushed inside the barrel, of 11 mm with thickness reduction (rd) of 7 mm, the contact pressure distribution was uniform along the contact length and over the circular section of the rod, as shown in Fig. 6. The pressure was distributed almost equally along the anchor which was undesirable in the anchor design. In order to minimize the occurrence of stress concentration, a further reduction of the barrel thickness near the loading end of the rod is required to minimize the confinement and consequently the contact pressure at this end. Fig. 7 shows the contact pressure distribution in anchor with (rd = 7 mm) on the external rod and wedge surfaces for presetting distances of 2 and 11 mm. For a presetting distance of 2 mm, the wedge was in full contact with the barrel due to the geometrical similarity of the wedges and the barrel which resulted in uniform contact pressure distribution along the rod.

The contact pressure distribution over the rod for different barrel thickness reduction (rd) values of 0, 7, and 15 mm and presetting distances of 2, 5, 8 and 11 mm were investigated. When the barrel thickness was not reduced (rd = 0), the contact pressure was higher at the loading end of the rod than that of the free end due to the larger barrel thickness at this end imposing higher restriction (confinement) of the wedges, as shown in Fig. 8. As the barrel thickness was reduced by 7 mm, the contact pressure slightly decreased (Fig. 9). Further barrel thickness reduction of 15 mm was made. The contact pressure decreased providing better contact pressure distribution that reduced the stress concentration (Fig. 10). A comparison between the contact pressure of the three barrel reduction values for an inserting distance of 2 and 11 mm is shown in Fig. 11. At an inserting distance of 2 mm, a little difference was observed between the three anchors as a result of the low confinement produced by the barrel on the wedges and consequently on the rod. With the wedges inserted by 11 mm, the contact pressure was
decreased significantly when the barrel thickness was reduced by 15 mm. However, a reduction of 7 mm had little effect on the contact pressure.

Tensile Loading

Anchors of thickness reduction of 7 and 15 mm were investigated under tensile loading. The wedge was preset into the barrel by 8.0 mm before pulling the loading end of the rod by 0.25 mm in each loading step. The load-displacement relationship of the rod, sleeve and wedge at the free end were examined and were found to move together without any relative displacement (Fig. 12). The anchor with (rd) of 15 mm had lower tensile load than that with (rd) of 7 mm due to the lower shear stress on the surface as a result of lower contact pressure.

The shear stresses on the rod-sleeve and wedge-barrel surfaces of these two anchors were calculated. Generally, the shear stress was higher on the rod-sleeve surface than the wedge-barrel surface due to the higher coefficient of friction of 0.24 at the rod-surface in compare to (0.0-0.02) at the wedge-barrel surface.

Fig. 13 shows the shear stress distribution in the anchor with thickness reduction of 7 mm and for pulling distance of the loading end of the rod of 0.25 and 1.75 mm. Since wedges, sleeve and rod were moving together, therefore the pulling distance increased the inserting distance of the wedge inside the barrel. This resulted in higher contact pressure and higher shear stress. In the anchor with (rd) of 15 mm, lower shear stress was produced because of the lower contact pressure (Fig 14).

Experimental Tests

A CFRP rod of 9.4 mm diameter and 1000 mm long was tested under tensile loading using an anchor with a radius reduction of 7 mm at one end and a reusable clamped anchor at the other. The specimen was prepared by cleaning the rod, and anchor components using acetone. After inserting the rod into the sleeve, the wedges were distributed evenly around the sleeve and held using a rubber band. Lubricant was applied to the outer surface of the wedges in order to facilitate their movement inside the barrel. The rod, sleeve and wedges were inserted inside the barrel using a hydraulic jack attached to steel frame.

At the other end, the clamped anchor was attached to the rod. The anchor consisted of a sleeve and two steel plates with a longitudinal semi circular groove to house the sleeve and rod. The clamping load was applied to the rod through a number of bolts.

The specimen was mounted on loading machine. The anchors were supported by two cross bearing plates. To measure the displacement of the anchor components, a linear variable differential transducer (LVDT) was attached to the rod at the test anchor end. More details are given in the literature.
For a presetting distance of 4 mm, the load-displacement of the rod was monitored. The rod failed prematurely at around 115 kN (80% of the ultimate strength of the rod). The main cause of the failure was a stress concentration at the loading end of the rod. Post test inspection of the failed specimen showed that the failure initiated at the location of threads on the inner surfaces of the wedge. The experimental data was compared to the finite element results, as shown in Fig. 15.

Therefore, smooth inner surface of the wedges and further thickness reduction of the barrel are recommended to avoid any stress concentrations. In addition, as shown in Fig. 11, reducing the barrel thickness by 7 mm had minimum effect on the contact stress distribution on the loading end of the rod.

**SUMMARY**

Variable thickness anchor systems with different values of barrel thickness reduction (rd = 0, 7, 15 mm) were modeled using ABAQUS finite element package. Contact pressure on rod surface was calculated for each anchor with presetting distance of 2, 5, 8 and 11 mm.

As the inserting distance increased, the contact pressure increased. The highest contact pressure was found in the anchor with no barrel thickness reduction which may lead to a premature failure of the rod due to the stress concentration at its loading end. However, at the maximum thickness reduction of 15 mm, the contact pressure decreased significantly near the loading end of the rod which contributed significantly to the avoidance of any stress concentration.

Tensile loading was also applied in the form of pulling distance of the rod loading end. After presetting the wedges by 8 mm inside the barrel, the rod was pulled by 0.25 mm in each step. The tensile load-displacement relationship was found for anchors with thickness reduction of 7 and 15 mm. Due to the lower contact pressure on the anchor of 15 mm thickness reduction, less tensile load was carried by the rod for the same pulling distance of the anchor with 7 mm thickness reduction. The shear stress on the rod-sleeve surface was higher than that on the wedge-barrel surface due to differences in coefficient of friction. Higher shear stress was noticed on the anchor with (rd) of 7 mm relating to the higher contact pressure on the rod surface. A comparison between the experimental and numerical results showed the similar behavior using both methods.

**References**


TABLE 1 — Properties of Anchor Components

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Figure 1—Anchor systems used with composite rods.

Figure 2—Wedge deformation outside the barrel of variable barrel thickness anchor.
Figure 3—Variable thickness barrel anchors: a) photo; b) dimensions; and c) sleeve (dimensions in mm).

Figure 4—Meshing schemes of the anchors.
Figure 5—Anchor parts and boundary conditions.
Figure 6—Contact pressure distribution on rod surface for anchor with $rd = 7\, \text{mm}$ and $11\, \text{mm}$ presetting distance.
Figure 7—Contact pressure distribution on the rod and external wedge surfaces in anchor with \( rd = 7 \) mm and presetting distance of a) 2 and b) 11 mm.

Figure 8—Contact pressure distribution on the rod of anchor with no thickness barrel reduction for different presetting distances.
Figure 9—Contact pressure distribution on the rod of anchor with thickness barrel reduction of 7 mm for different presetting distances.

Figure 10—Contact pressure distribution on the rod of anchor with thickness barrel reduction of 7 mm for different presetting distances.
Figure 11—Contact pressure distribution on the rod of anchor with thickness barrel reductions of 0, 7, 15 mm for presetting distances of 2 and 11 mm.

Figure 12—Tensile load-displacement relationship of anchors with (rd) of 7 and 15 mm and presetting distance of 8 mm.
Figure 13—Shear stress distribution on rod-sleeve and wedge-barrel surfaces in anchor with (rd = 7 mm) and pulling distance of a) 0.25 and b) 1.75 mm.

Figure 14—Shear stress distribution on rod-sleeve and wedge-barrel surfaces in anchor with (rd = 15 mm) and pulling distance of a) 0.25 and b) 1.75 mm.

Figure 15—Tensile load-displacement relationship of anchors with (rd) of 7 and presetting distance of 4 mm.
Behavior of Pretensioned Type II AASHTO Girders Constructed with Self-Consolidating Concrete

by H.R. Hamilton III, T. Labonte, and M.H. Ansley

Synopsis: Self-consolidating concrete (SCC) is a relatively new approach to making concrete and is characterized by its high flowability and resistance to aggregate segregation in the plastic state. SCC has become a popular alternative for commercial precast elements and is being evaluated for use in precast bridge girders. This paper outlines structural testing of six precast, pretensioned, AASHTO Type II girders. Three were constructed using SCC, and three using a conventional mix. The major tasks included performing plastic and hardened property tests, constructing SCC beams without vibrating, determining the prestress transfer length, monitoring the camber, and finally testing the beams in such a manner as to produce flexure and shear dominated failure modes. Results of the construction and testing are presented.

Keywords: bridge girder; precast; prestressed; self-consolidating concrete
INTRODUCTION

BACKGROUND

Self-consolidating concrete (SCC) is a highly workable, non-segregating concrete that does not require mechanical vibration during placement. SCC evolved out of underwater concrete admixture technology in Japan in the 1980’s and the desire to make the casting process more efficient due to a low skilled labor supply (Okamura 1996). Several European countries adopted the use of SCC starting in the early 1990’s, and have successfully constructed many bridges, buildings, and other concrete structures using this material. In the United States, SCC is currently being used mainly in the precast industry for the construction of non-critical structural components and other products that are not highly stressed elements of major concrete structures. Several pedestrian bridges have been constructed with SCC in the US, and there have been successful applications of SCC in building construction.

SCC can flow to fill areas around dense reinforcement and through thin openings under its own weight with minimal formation of voids, segregation or water bleed (PCI 2003). Additional advantages include eliminating mechanical vibration, improving formed surface finishes, reducing finishing time, improving labor force efficiency, improving working conditions, and safety. The workability of SCC is better than the highest class of workability associated with normal high-performance concrete typically used in precast concrete fabrication plants (PCI 2003). Standards currently being developed define a concrete mix as SCC when the mix meets quantifiable workability criteria based on its confined flowability, passing ability, and resistance to segregation. The highly flowable properties have stimulated the development of several new plastic property tests that are applicable only to SCC.

RESEARCH APPROACH

This project had several phases. Initially the FDOT State Materials Office conducted mix designs in cooperation with the precast supplier. Plastic and hardened properties of the SCC were tested in both the laboratory and field trial mixes. Once the mix design had been completed, the precast supplier fabricated the girders using the design mix. FDOT Structures Research Center and University of Florida personnel instrumented the girders to determine prestress transfer length. The girders were then shipped to the FDOT Structures Research Center where they were monitored for camber growth for approximately 200 days. At the conclusion of the camber monitoring period, the beams were tested in shear or flexure to determine the structural capacity and behavior.
INTRODUCTION

Trial mixes were used to obtain mix designs with the targeted fresh properties. Six 42-foot AASHTO Type II beams were constructed and samples were taken from the casting mix to be used for material testing. Three of the six beams were constructed with SCC, and three beams were constructed with a typical approved mix. Instrumentation was installed on the beams before the transfer of prestress to measure the transfer lengths. Camber monitoring started immediately after the transfer of prestress and continued for approximately 200 days from casting. All beams were then tested to destruction using short shear spans. The distance between the bearing and beam end was varied to encourage strand slip at ultimate capacity.

SCC MIX DEVELOPMENT AND MATERIALS TESTING

An FDOT Class VI mix with a target 28-day concrete compressive strength of 8,500 psi was used as a template for the development of conventional (hereinafter referred to as standard) and SCC concrete mixes, respectively. This work was conducted by the FDOT State Materials Research Office and is detailed in Labonte and Hamilton 2005.

The trial mixes were batched and tested to determine the optimum mix design and to determine if adjustments to the relative constituent quantities were necessary. The same relative quantities of cement, fly ash, and water were used for both the standard and SCC mixes. SCC properties were obtained by using a larger dosage of high-range water reducing (HRWR) admixture with little change to other constituent volumes.

Cylinders were taken from the mix used to cast the beams for later compressive and tensile strength testing. One set of cylinders was shipped to the FDOT State Materials Office for storage and testing. The results of these tests are shown in Figure 1. The cylinders for these compressive test results were moist cured and tested at typical ages. An additional two cylinders per beam were transported with each beam to the FDOT Structures Laboratory and tested when the respective beam was tested (Table 1). These cylinders were cured and stored with the beams until testing. Beam testing occurred approximately 8 to 10 months after the beams were constructed compared to the early age testing of the cylinders shown in Figure 1. The SCC tested above 9,000 psi, while two of the three standard mixes tested at approximately 7,500 psi. One of the standard mix beams, however, tested over 10,000 psi. It is not clear why there was such a difference in the standard mix strengths. In addition, tensile tests were conducted using both spilt cylinder and beam test configurations (Table 2).

BEAM DESIGN AND CONSTRUCTION

The objectives of the beam testing were to compare the strand transfer length, camber growth, and structural properties of the beams. A total of six beams were constructed for this testing. Four of the beams (two SCC and two standard) were designed to be tested in flexure and shear with a composite cap to simulate the composite
action of the bridge deck. Two (one SCC and one standard) were designed to be tested in shear without the benefit of the composite action from the deck. These specimens also had light shear reinforcement at the ends to determine if there was any difference in shear behavior between the SCC and standard concrete.

The AASHTO Type II beam tendon size and configuration were designed to meet the requirements of the AASHTO LRFD Bridge Design Specifications for a fictitious bridge in which the beams were assumed to be spaced at 6 ft with a 40-ft span. The tendon was composed of twelve 0.5-in. diameter ASTM A416 Gr 270 prestressing strands. Two strands were debonded for a length of 6 ft. The deck thickness was assumed to be 10 inches. Florida Department of Transportation (FDOT) software (LRFD P Beam Version 1.85) was used to design the beam. The beam details are shown in Figure 2 through Figure 4.

The flexural beams were tested with a composite concrete top flange (referred to as “cap”) to model the compression area provided by the bridge deck in actual service conditions (Figure 5). Although the design called for a beam spacing of 6 ft., laboratory conditions allowed a maximum width of 2-ft. The cap was constructed by FDOT Structures Laboratory personnel prior to testing using a Class II (Bridge Deck) ready-mix concrete ($f'_c = 4,500$ psi). Cylinder strengths for the cap concrete are given in Table 1. The compressive strengths were relatively consistent with the exception of the cap for SCCF2.

The stirrup spacing away from the end region of the non-capped beams was set at four feet to promote diagonal cracking and strand slip. This approach was used as a means to compare the behavior changes resulting from the use of SCC. A typical FDOT prescribed arrangement of mild steel reinforcement was included in the end region of the non-capped beams to force the failure location to the area of minimal shear reinforcement (Figure 3). The first five stirrups were double leg with the remaining stirrups single leg.

The capped beams included a conventional arrangement of transverse reinforcement, which met the requirements of the Design Specifications. This included a stirrup spacing away from the end region of the capped beams of 12 in. The first five stirrups in the capped beam were double leg with the remaining stirrups single leg.

The six 42-foot long AASHTO Type II beams were cast in a single day at a prestressed concrete plant in Jacksonville, Fla. FDOT quality assurance personnel were present to ensure the beams were cast using correct procedures and met specified tolerances. To eliminate a vibration carry-over effect from the consolidation of the standard concrete due to the continuously connected forms, all standard concrete beams were cast and consolidated before the SCC beams were poured. No consolidation was utilized on the SCC beams.

Prestress Transfer

The beams were cast in a single line on a single casting bed. The transfer of prestress was accomplished by torch-cutting single strands simultaneously between alternate pairs of beam ends as shown in Figure 6. Cuts were also made at each end of the casting bed, resulting in each beam having one end in which the strands were released suddenly. This method of release is quite abrupt and has been shown to result in longer transfer lengths than a gradual release (Russell and Burns 1997).
Due to low early concrete strengths in both the SCC and standard beams and scheduling conflicts, the prestress transfer was delayed until fifteen days after casting. The five-day cylinder compressive strengths from the precasting plant were 3170 psi for the standard concrete and 3810 psi for the SCC. It is not known why the early strengths were low. One explanation is that the cement used to produce the concrete was changed, which occurred after the verification mix but before the beam mix.

Before the transfer of prestress, strain gauges were installed on the bottom flange of each end of one standard beam and one SCC beam to determine transfer lengths. Camber was also monitored from transfer of prestress to approximately 200 days after casting. Details of camber and transfer testing and results can be found in Labonte and Hamilton 2005.

**STRUCTURAL TESTING**

All beams were tested in three-point loading as shown in Figure 7 with the distances specified in Table 3. Both ends of each beam were tested by placing the middle loading point close to the respective end. The testing program was designed to compare the structural behavior of SCC beams with standard beams, including ultimate load, deflection at the ultimate load, measured to theoretical capacity ratio, strand slip, and web cracking load. The focus of the testing was on the flexural and shear failure modes. The shear span-to-depth ratio (a/d) was varied among the test configurations to force either a flexural or shear failure mode, as indicated in Table 3. The distance from the end of the beam to the center of the support was reduced in some specimens to promote strand debonding and slip. In subsequent discussions, each test is referred to by the designation given in Table 1. For instance S1-SCCS is the shear test conducted on the north end of beam SCCS.

Two of the six beams were constructed and tested to investigate the shear behavior of the SCC beam as compared to the standard beam (Shear in Table 3). The wide stirrup spacing previously discussed in SCCS and STDS was used to minimize the influence of the transverse steel on the performance of the beams. The shear test geometry was intended to create a concrete strut or node crushing failure mode, along with the possibility of some bonded strand slip. Two beams were constructed and tested to investigate the structural behavior of the SCC beam as compared to the standard beam in a condition of combined shear and flexure (Flexure in Table 3). The intent was that the geometry of the shear-flexure tests would cause a flexural failure mode with considerable shear cracking. Finally, two of the beams were tested with both a short shear span and a short available development length (Strand-slip in Table 3). The short development length was accomplished by positioning the beam on the support such that there was a six-inch overhang beyond the bearing. This bearing placement was used to promote a strand slip failure mode.

The test setup and instrumentation for the shear test is shown in Figure 8. Loading was held at a rate of 0.15 kips per second. Shear-flexure and Shear-slip test setups and instrumentation were similar. See Labonte and Hamilton 2005 for further details.
Behavior of the beams was uneventful until web cracking formed suddenly when the load was between approximately 110 kips to 120 kips in both standard and SCC specimens. All four shear tests resulted in a single crack with a common angle of approximately 30° (from horizontal) for each beam (Figure 9). The load versus deflection remained linear until the loads were between approximately 155 kips and 220 kips (Figure 10). The load versus deflection for all beams reached a plateau and had a very short inelastic range with failure loads between 180 kips and 230 kips with approximately one inch of deflection preceding a drastic loss in load. The failure mode for all four shear tests was a compression failure in the top flange under and adjacent to the load point.

Table 4 shows the peak loads and maximum deflections. The maximum deflection was measured under the load and was the deflection just before the sudden loss in load. The S1-STDS beam performed slightly better with an 8.7% greater load carrying capacity than the S1-SCCS beam. The S1-STDS beam also deflected approximately 22% more than the S1-SCCS beam. The S2-SCCS and S2-STDS beams had nearly identical failure loads and deflections.

Figure 11 shows the strand slip pattern for S2-SCCS in which immediate movement of the debonded strands was detected (See Figure 2 for location of debonded strands). This movement is expected due to the flexural tensile strains that accumulate between the end of the beam and the point at which the strands are bonded. The strand is not bonded so the concrete moves relative to the strand creating the displacement shown in the graph. No movement in the fully bonded strands was detected. Only in test S2-STDS was slip noted in the instrumented fully bonded strands (Figure 12). In that test, strands 1 and 2 clearly began displacing at a load of 195 kips. The SCC beam reached a slightly higher ultimate capacity than the standard beam, and the SCC beam reached this capacity without any strand slip. The strand movements of the sheathed strands were approximately the same for the SCC beams and the standard beams. The bonded strand displacement with the standard beam may be explained by considering the results from the transfer of prestress. There was a large amount of strand movement at the end of the beam during the transfer of prestress. This large amount of strand movement during the prestress transfer may have contributed to the observed strand slip during the structural testing of the standard beam. Comparison of the shear test data revealed little difference between the performance of SCC and standard concrete for this series of testing.

The aggregate distribution at the shear crack in SCCS girder was examined. Figure 13 shows the face of the crack after removal of the beam end. Although no quantitative measurements were made, there appeared to be less aggregate in the top flange than at the bottom. Inspection of the crushed top portion of the beam at the opposite end, however, indicated that more aggregate remained in the top of the beam at that location.
Web cracking formed in the beams at a load in the range of 144 kips to 157 kips under a constant load application rate. The load-deflection response remained linear until a load of approximately 200 kips (Figure 14). The load versus deflection for all beams reached a plateau typical of a flexure dominated failure mode. The peak loads ranged between 248 kips and 263 kips with 1.6 to 3.1 inches of deflection preceding a sharp drop in load. In all four shear-flexure tests, the composite cap failed in compression due primarily to flexural compressive stress at the location directly under the load point.

Figure 15 shows the F2-SCCF1 beam and F2-STDF2 beam following failure and unloading. Both beams had typical flexural tensile cracks emanating from the bottom of the beam under the load point. Visual observation indicated comparable cracking patterns and widths in the SCC and standard specimens. Due to the short distance between the load point and support, the beams experienced high shear stresses simultaneously with the high flexural stresses, causing a considerable number of diagonal cracks to form in contrast to the shear tests in which a single shear crack formed. The shear crack spacing and widths visually appeared similar for both SCC and standard beams.

Table 5 shows the maximum loads and deflections for the four shear-flexure tests. The beam deflections were measured at the load point. F1-SCCF1 performed slightly better with a 6.0% greater load carrying capacity than the F1-STDF2 beam. F1-STDF2 outperformed F1-SCCF1 in terms of ductility with a 41.5% greater deflection immediately preceding failure. The F2-STDF2 beam performed slightly better than the F2-SCCF1 beam with a 1.6% greater load carrying capacity. The F2-STDF2 beam also outperformed the F2-SCCF1 beam in terms of ductility with a 9.3% greater deflection. In summary, the flexural capacity of the SCC and standard beams were comparable, but the displacement ductility of the standard beam was slightly better than that of the SCC beam.

Figure 16 shows the displacements of two of the strands during the F1-SCCF1 shear-flexure test. No strand movement of the fully bonded strands (L1) was detected in any test. The sheathed strand (L5), however, began to slip at a load of approximately 200 kips leading to the plateau in the load-displacement relationship. Based on the relatively long plateau noted in the load-displacement curve the flexural capacity of these specimens was likely controlled initially by the bond and yield strength of the prestressing strands and ultimately by the compressive strength of the cap. As the load increased beyond the elastic range, the strands begin to yield or slip, or both. This behavior is apparent when comparing the capacities of the beam ends adjacent to the cut locations with those away from the cut locations. The abrupt release of the strands near the anchorage zone at the end of the beam lengthens both the transfer and development length of the strands. The longer free length of the unbonded strands will likely enhance this effect in the debonded strands because of the increase energy released when cut. Consequently, the beam ends where the strands were cut had slightly lower flexural capacities than those that were away from the release point. This is true regardless of whether the beam was SCC or standard mix.
The four shear-slip tests are compared in Figure 17. Initial behavior was linear up to web cracking, which occurred between 124 and 134 kips. Even beyond initial cracking the response remained linear until a load of approximately 220 kips was reached. The load versus deflection for all beams reached a plateau and had a failure load range of 269 kips to 314 kips with 0.9 to 1.2 inches of deflection preceding a sharp loss in load. With the exception of SS2-SCCF2 the failure mode was compression failure of the composite cap. SS2-SCCF2 failed prematurely due to strand slip at the beam end. Figure 18 shows the typical failure mode.

Table 6 shows the maximum loads and deflections for all four shear-slip tests. Figure 17 is a plot of the deflections that occurred over the full range of loads. The SS1-SCCF2 beam was 18% more ductile than the SS1-STDF1 beam in terms of the beam deflection immediately before failure. SS2-STDF1 had nearly twice the maximum deflection as that of SS2-SCCF2, which was likely due to the strand slip in the latter specimen (discussion to follow).

Table 7 shows the deflections near the end of the linear regions of the load-deflection curves for each beam. SS1-SCCF2 and SS1-STDF1 resulted in nearly equal deformations at 218 kips. The SS2-STDF1 beam had 17% more deflection than the SS2-SCCF2 beam at 228 kips.

Figure 19 shows the strand displacement immediately before failure for the fully bonded strands in the bottom row of the strand arrangement. There was minimal slip in both beam types with the SS1-SCCF2 beam performing slightly better than the SS1-STDF1 beam. The SS2-STDF1 beam performed much better than the SS2-SCCF2 beam in the second test due to the change in failure modes likely caused by a difference in the prestress transfer method. The SS2-SCCF2 beam end was at the cut end during the prestress transfer, and the SS2-STDF1 beam end was at the free end during prestress transfer as shown in Figure 6. Consequently, strand slip in SS2-SCCF2 caused failure at a lower load (Figure 19). The measured transfer lengths (reported in Labonte and Hamilton 2005) consistently indicated that the transfer lengths adjacent to the release points were longer. The large impact force of the quick release technique used may have produced sufficient permanent deformation to allow the SCC beam to fail by strand slip. Additionally, the SCC beam moved several inches during the prestress transfer due to the shock of unsynchronized release. STDF1, however, did not move during the prestress transfer.

**SUMMARY AND CONCLUSIONS**

Six AASHTO Type II pretensioned girders were constructed, three using SCC and three using a conventional (referred to as “standard”) concrete mix typically used by FDOT in AASHTO girders. Plastic and hardened properties of mix samples were tested. Structural testing to destruction was performed with the target failure modes being shear and flexure. The following can be concluded from this testing program:

- Reduced construction time, improved labor efficiency, reduced noise, and improved safety were noted during the construction of the SCC beams.
In general, the compressive strength of SCC is expected to be higher than that of a corresponding standard concrete. This was found to be true in one of the two trial mixes prepared during the mix design phase. The other trial mix SCC compressive strengths were lower. SCC used to fabricate the beams was also found to have lower early age compressive strength than the standard concrete. Compressive strengths of the two concretes converged to approximately 8,800 psi at 56 days. Cylinders tested at the time of the beam tests indicated higher compressive strengths for the SCC in most cases. It was not clear why these differences in compressive strengths were noted. Differences in cylinder curing conditions (between the cylinders that accompanied the beams and those that were tested in the materials laboratory) may have contributed. In addition, the cement supplier was changed between the trial mix and beam construction, which may have contributed to the variation.

No notable differences were found in flexural capacity, shear capacity, or observed web cracking (during load testing) between the SCC and standard beams. One exception was the fully bonded strand slip in SS2-SCCF2, which resulted in a 15% lower ultimate capacity for SCC. It is believed that, based on the transfer length measurements, the abrupt prestress transfer conditions may have contributed to the early slip. Indeed, the prestress transfer conditions may have accounted for more variation in the beam performance than the difference in the type of concrete.

Total deflections measured during the load tests indicated that the standard mix had slightly better ductility than SCC with the standard beams reaching an average of 17.1% more deflection than the SCC beams at the ultimate load.

Shear dominated failure modes were observed to consist of either a compression failure in the top of the section (either the top of the precast or topping, depending on the test specimen) near the point of applied load or, in one case, strand slip at the support. This is typical strut and tie behavior that has been commonly observed in previous research.

Some aggregate segregation was noted in one of the SCC beams, but there was no indication of widespread problems.

**ACKNOWLEDGMENTS AND DISCLAIMER**

The authors would like to acknowledge and thank the Florida Department of Transportation (FDOT) for providing the funding for this research project. This project was a collaborative effort among the University of Florida, FDOT Structures Research Laboratory (Tallahassee) and the FDOT State Materials Office (Gainesville). Mix development and materials testing was conducted by Charles Ishee, Mario Paredes, Richard Delorenzo and Charlotte Kasper at FDOT State Materials Office. Structural testing was conducted by the FDOT Structures Research Laboratory (David Allen, Frank Cobb, Steve Eudy, Tony Johnston, Paul Tighe). The authors would also like to thank Gate Concrete Products Company Jacksonville, Fla., and Jim Kunberger and Bruce Hunter for constructing the specimens and Eckart Buehler with Master Builders for
technical assistance with the mix designs. Finally, the authors would like to thank
Ghulam Mujtaba, FDOT State Materials Office, for his input and technical expertise in
developing and conducting this research.
The opinions, findings, and conclusions expressed in this publication are those
of the authors and not necessarily those of the State of Florida Department of
Transportation.

REFERENCES

AASHTO, LRFD Bridge Design Specifications, 2nd Edition, American Association of

Labonte, T., and Hamilton III, H. R., “Self-Consolidating Concrete (SCC) Structural
Investigation,” Florida Department of Transportation Project Report BD545#21,

Okamura, H., “Self-Compacting High-Performance Concrete,” Concrete International,

Precast/Prestressed Concrete Institute, “Interim Guidelines for the Use of Self-
Consolidating Concrete In Precast/Prestressed Concrete Institute Member Plants,”

Russell, B.W., and Burns, N.H., “Measurement of Transfer Lengths on Pretensioned

Table 1. Compressive strength results from cylinders tested on or near date of beam test.

<table>
<thead>
<tr>
<th>Test-Beam</th>
<th>Cylinder 1 (psi)</th>
<th>Cylinder 2 (psi)</th>
<th>Average Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCCF1</td>
<td>9,430</td>
<td>8,760</td>
<td>9,090</td>
</tr>
<tr>
<td>SCCF2</td>
<td>10,700</td>
<td>11,100</td>
<td>10,900</td>
</tr>
<tr>
<td>SCCS</td>
<td>9,910</td>
<td>10,200</td>
<td>10,030</td>
</tr>
<tr>
<td>STDF1</td>
<td>10,600</td>
<td>10,540</td>
<td>10,580</td>
</tr>
<tr>
<td>STDF2</td>
<td>7,460</td>
<td>7,850</td>
<td>7,660</td>
</tr>
<tr>
<td>STDS</td>
<td>7,220</td>
<td>7,760</td>
<td>7,490</td>
</tr>
<tr>
<td>SCCF1 Cap</td>
<td>7,890</td>
<td>7,600</td>
<td>7,740</td>
</tr>
<tr>
<td>STDF2 Cap</td>
<td>7,520</td>
<td>7,230</td>
<td>7,370</td>
</tr>
<tr>
<td>SCCF2 Cap</td>
<td>8,990</td>
<td>8,940</td>
<td>8,960</td>
</tr>
<tr>
<td>STDF1 Cap</td>
<td>7,920</td>
<td>6,970</td>
<td>7,440</td>
</tr>
</tbody>
</table>
# Table 2. Twenty-eight day tensile strength test comparison

<table>
<thead>
<tr>
<th>Test</th>
<th>Average Tensile Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Standard</td>
</tr>
<tr>
<td>Split cylinder (ASTM C496)</td>
<td>815</td>
</tr>
<tr>
<td>Beam (ASTM C78)</td>
<td>900</td>
</tr>
</tbody>
</table>

# Table 3. Test setup geometry

<table>
<thead>
<tr>
<th>Test-Beam</th>
<th>Target Failure mode</th>
<th>Location*</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>a/d</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-SCCS</td>
<td>Shear</td>
<td>North</td>
<td>6'-0&quot;</td>
<td>28'-6&quot;</td>
<td>1'-6&quot;</td>
<td>2.25</td>
</tr>
<tr>
<td>S2-SCCS</td>
<td></td>
<td>South</td>
<td>5'-0&quot;</td>
<td>28'-6&quot;</td>
<td>1'-0&quot;</td>
<td>1.88</td>
</tr>
<tr>
<td>S1-STDSDS</td>
<td></td>
<td>North</td>
<td>6'-0&quot;</td>
<td>28'-6&quot;</td>
<td>1'-6&quot;</td>
<td>2.25</td>
</tr>
<tr>
<td>S2-STDSDS</td>
<td></td>
<td>South</td>
<td>5'-0&quot;</td>
<td>28'-6&quot;</td>
<td>1'-0&quot;</td>
<td>1.88</td>
</tr>
<tr>
<td>F1-SCCFS1</td>
<td>Flexure</td>
<td>North</td>
<td>9'-2&quot;</td>
<td>21'-2&quot;</td>
<td>1'-0&quot;</td>
<td>2.50</td>
</tr>
<tr>
<td>F2-SCCFS1</td>
<td></td>
<td>South</td>
<td>10'-0&quot;</td>
<td>28'-0&quot;</td>
<td>1'-0&quot;</td>
<td>2.73</td>
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<tr>
<td>F1-STDFS2</td>
<td></td>
<td>North</td>
<td>9'-2&quot;</td>
<td>21'-2&quot;</td>
<td>1'-0&quot;</td>
<td>2.50</td>
</tr>
<tr>
<td>F2-STDFS2</td>
<td></td>
<td>South</td>
<td>10'-0&quot;</td>
<td>28'-0&quot;</td>
<td>1'-0&quot;</td>
<td>2.73</td>
</tr>
<tr>
<td>SS1-SCCFS2</td>
<td>Strand Slip</td>
<td>North</td>
<td>6'-6&quot;</td>
<td>28'-6&quot;</td>
<td>6&quot;</td>
<td>1.63</td>
</tr>
<tr>
<td>SS2-SCCFS2</td>
<td></td>
<td>South</td>
<td>6'-0&quot;</td>
<td>28'-0&quot;</td>
<td>6&quot;</td>
<td>1.63</td>
</tr>
<tr>
<td>SS1-STDSDS</td>
<td></td>
<td>North</td>
<td>6'-6&quot;</td>
<td>28'-6&quot;</td>
<td>6&quot;</td>
<td>1.76</td>
</tr>
<tr>
<td>SS2-STDSDS</td>
<td></td>
<td>South</td>
<td>6'-0&quot;</td>
<td>28'-0&quot;</td>
<td>6&quot;</td>
<td>1.63</td>
</tr>
</tbody>
</table>

* Location of beam end in casting bed.

# Table 4. Maximum loads and deflections for shear tests

<table>
<thead>
<tr>
<th>Test-Beam</th>
<th>Maximum Load (kips)</th>
<th>Release Point*</th>
<th>Maximum Deflection (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-SCCS</td>
<td>178</td>
<td>No</td>
<td>0.96</td>
</tr>
<tr>
<td>S1-STDSDS</td>
<td>193</td>
<td>Yes</td>
<td>1.17</td>
</tr>
<tr>
<td>S2-SCCS</td>
<td>231</td>
<td>Yes</td>
<td>1.17</td>
</tr>
<tr>
<td>S2-STDSDS</td>
<td>229</td>
<td>No</td>
<td>1.11</td>
</tr>
</tbody>
</table>

* Yes indicates this beam was adjacent to a release point during prestress transfer.

# Table 5. Maximum loads and deflections for shear-flexure tests

<table>
<thead>
<tr>
<th>Test-Beam</th>
<th>Maximum Load (kips)</th>
<th>Release Point*</th>
<th>Maximum Deflection (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1-SCCFS1</td>
<td>263</td>
<td>Yes</td>
<td>1.64</td>
</tr>
<tr>
<td>F1-STDFS2</td>
<td>248</td>
<td>No</td>
<td>2.32</td>
</tr>
<tr>
<td>F2-SCCFS1</td>
<td>250</td>
<td>No</td>
<td>2.80</td>
</tr>
<tr>
<td>F2-STDFS2</td>
<td>254</td>
<td>Yes</td>
<td>3.06</td>
</tr>
</tbody>
</table>

* Yes indicates this beam was adjacent to a release point during prestress transfer.
Table 6. Maximum loads and deflections for shear-slip tests

<table>
<thead>
<tr>
<th>Test-Beam</th>
<th>Maximum Load (kips)</th>
<th>Release Point*</th>
<th>Maximum Deflection (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS1-SCCF2</td>
<td>290</td>
<td>No</td>
<td>1.04</td>
</tr>
<tr>
<td>SS1-STDF1</td>
<td>293</td>
<td>Yes</td>
<td>0.88</td>
</tr>
<tr>
<td>SS2-SCCF2</td>
<td>269</td>
<td>Yes</td>
<td>0.61</td>
</tr>
<tr>
<td>SS2-STDF1</td>
<td>314</td>
<td>No</td>
<td>1.21</td>
</tr>
</tbody>
</table>

*Yes indicates this beam end was adjacent to a release point during prestress transfer

Table 7. Load and deflections at end of elastic region for shear-slip tests

<table>
<thead>
<tr>
<th>Test-Beam</th>
<th>Load (kips)</th>
<th>Deflection (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS1-SCCF2</td>
<td>218</td>
<td>0.26</td>
</tr>
<tr>
<td>SS1-STDF1</td>
<td>218</td>
<td>0.25</td>
</tr>
<tr>
<td>SS2-SCCF2</td>
<td>228</td>
<td>0.24</td>
</tr>
<tr>
<td>SS2-STDF1</td>
<td>228</td>
<td>0.28</td>
</tr>
</tbody>
</table>

Figure 1. Beam mix average cylinder strength comparison.

Figure 2. Prestressed beam design detail (a) end section (b) middle section.
Figure 3. Non-capped beam elevation detail.

Figure 4. Capped beam elevation detail.

Figure 5. Composite cap detail.
Figure 6. Specimen configuration in prestressing bed. Cut indicates release points where strands were torch cut during release.

Figure 7. Three point loading test geometry.

Figure 8. Shear test setup and instrumentation.
Figure 9. Failed S2-STDS beam.

Figure 10. Comparison of beam behavior during shear tests. Deflection measured under load point.

Figure 11. Strand slip pattern for S1-SCCS. This pattern was typical for S1-STDS and S1-SCCS.
Figure 12. Strand slip for S2-STDS.

Figure 13. SCCS girder after shear testing. Note higher density of aggregate in the bottom of beam than in the top.
Figure 14. Comparison of beam behavior during shear-flexure tests. Deflection measured under load point.

Figure 15. Failed F2-SCCF1 beam.

Figure 16. Strand slip for F1-SCCF1. Typical for F2-STDf2, F2-SCCF1, and F1-STDf2.
Figure 17. Comparison of beam behavior during shear-slip tests. Deflection measured under load point.

Figure 18. Failed SS2-STDF1 beam.

Figure 19. Typical fully bonded strand slip for SS2-SCCF2.
Professor Ned H. Burns, Ph.D.—Scholar, Educator and Engineer

by R.W. Furlong, Ph.D., FACI, HonMASCE

Synopsis: Career highlights are presented regarding Ned H. Burns, now recognized as a “Legend of the Prestressed Concrete Industry.” Dr. Burns’ childhood, family, marriage, military service, and education are described briefly. His development in research and teaching is chronicled as awards began to accumulate. Leadership positions are indicated while awards continued in recognition of his excellence in structural engineering education and practice.

Keywords: educator; engineer; gentleman; scholar
Richard W. Furlong has been a colleague of Professor Burns on the Civil Engineering faculty at the University of Texas at Austin since 1958. Professor Furlong has delivered numerous seminars on designing concrete structures, and he has been a member of many ACI technical committees. He served the ACI Board of Directors 1977-1981.

**Professor Ned H. Burns, Ph. D. – Scholar, Educator and Engineer**

This certainly is an auspicious occasion for all who know Ned Burns. Lives there a person who has met Ned without thinking of him as a very good friend? I was very pleased to be invited by ACI Committee 423 to prepare a paper for presentation at the end of this Seminar acknowledging that Dr. Burns is indeed a Legend in the technology and practice of prestressed concrete and post-tensioned concrete.

Ned introduced himself to me about 47 years ago as I was unpacking a meager collection of books and papers during my first day on the campus of The University of Texas, when there was only one university by that name. It was not until 1966 that it became identified as The University of Texas at Austin. Ned’s enthusiasm in greeting me, the newest addition among his colleagues, was both pleasing and startling. He made it quite clear that he would do anything and everything he could to optimize whatever we could accomplish together in the new environment. His thoughtful openness has never waned since that hot summer day in 1958. Under the mentoring of Phil Ferguson, he helped teach Fundamentals of Reinforced Concrete during the next academic year before Ned left Austin to accept a Graduate Assistantship at the University of Illinois.

**Destined to be a Scholar**

Ned Hamilton Burns was the youngest among six children. He was born in Magnolia, Arkansas, where his father was the Superintendent of schools. As the son of the superintendent and with five older siblings coaching him, Ned’s response was positive as usual. After the family moved to Texarkana, a city that straddles the border between Arkansas and Texas, he completed secondary school education, graduating as Valedictorian from Texas High School. He completed a year of higher education at Texarkana Junior College before transferring to The University of Texas. In Austin, Ned was employed part-time in the Scottish Rite residence for women at the north edge of the university campus. One resident, Martha Fontaine, must have enjoyed special attention from one of the young men who delivered food and cleared dishes. In 1954, Ned earned his degree: Bachelor of Science in Architectural Engineering with Highest Honors.

Mosher Steel Company employed Ned as a steel detailer during the year 1954-55 before the United States Army claimed him during the period 1955–57. In Henderson, Texas late in the spring of 1955, Ned and Martha Fontaine were married. Ned was stationed at Fort Lewis, Washington, and the newlyweds lived in Tacoma. While living in Tacoma and visiting the plant of a local precast concrete fabricator, Ned was introduced to some applications of prestressing concrete by Dr. Arthur Anderson, a partner in the firm of Anderson, Birkland, Anderson and Mast. It might be said that Ned never recovered from that experience….nor desired to. The Army transferred Ned to its
Ned H. Burns Symposium

Engineering Research Lab, Fort Belvoir, Virginia before he returned to civilian life in 1957. Ned and Martha returned to The University of Texas, where Ned served as an Instructor in the Department of Civil Engineering. He was responsible for some courses in Drawing and in Materials Testing while working toward a Masters Degree and a thesis Efficiency of Spiral Reinforcement for Lapped Splices with Professor Phil Ferguson. His MS in Civil Engineering was awarded in 1958. A daughter, Kathleen, was born late in the summer of 1958.

A Research Assistantship for graduate study at the University of Illinois was accepted by Ned, and the Burns moved to Urbana, Illinois early in 1959. Talbott Laboratory with the University of Illinois during 1958 was recognized as North America’s leading institution for research in reinforced concrete. The participation of Ned Burns surely enhanced that recognition. While he was a graduate student in Illinois, Ned joined the American Concrete Institute, and he has been an ACI member ever since. A second daughter, Stephanie, was born in 1961. Ned’s dissertation, Moment-Curvature Behavior of Reinforced Concrete Beams in Flexure, supervised by Professor Chester Seiss, was completed and the degree Doctor of Philosophy was awarded in June 1962.

Teaching and Research Launched at The University of Texas at Austin

Dr. Burns accepted appointment as Assistant Professor of Civil Engineering at the University of Texas in 1962. The on-campus materials and testing laboratory was rather inadequate in size and in equipment for handling full-scale concrete test specimens. However, with the help and leadership of J. Neils Thompson, the facilities of an abandoned magnesium plant located 7 miles north of the Main Campus were made available to The University. Among assets of the former factory was a receiving area with a 24-ton crane and some 10,000 sq ft of concrete floor space, most of it without walls. Ned joined his colleagues in adapting “jury rigged” test facilities in that somewhat primitive environment while seeking explanations for how and why structures and components responded to almost any loading. Ervin S. Perry was his first Ph. D. candidate. His first dissertation supervision involved the use of very lightweight concrete as a non-resilient cushion for controlling the force-time impact from a falling mass.

Professor Ferguson had suggested that there was a need for a graduate course in prestressed concrete, and Dr. Burns developed such a course. He has maintained and improved that prestressed concrete course with knowledge gained from his supervision of laboratory experiments ever since. His initial experiments with prestressing concrete involved studies of moment-curvature behavior in pretensioned beam specimens with some strands partially unbonded. Subsequent studies were directed toward development of continuity at a joint between precast prestressed beams.

Each year some improvements were made in the facilities at the former receiving area of the magnesium plant. After The University gained official ownership of the land, walls were added and air conditioning was installed for lab offices. The facility today has been named the Ferguson Structural Engineering Laboratory.
Always the doors to Ned’s offices on campus and at the laboratory were open to students and colleagues seeking help, advice, or to share a humorous story. Dr. Burns was a good listener with patience adequate to determine the needs of frequent visitors. Within three years of his return to The University of Texas, in 1965, Ned’s pedagogical skill was acknowledged as he won the College-wide General Dynamics Award for Excellence in Teaching. The Civil Engineering Student Engineering Council designated Dr. Burns as the Distinguished Advisor for the Department in 1967 and again in 1970.

Dr. Burns was promoted to the rank of Associate Professor in 1965. He supervised development of computer programs for analytical methods to predict non-linear behavior of prestressed flexural members, and he initiated experimental studies of prestressed two-way floor systems with concurrent development of methods of analysis.

The third child of Ned and Martha, son Michael was born in 1967. Dr. Burns was promoted to the rank of Professor in 1972.

Throughout his career, Dr. Burns won numerous awards, published many papers, given several principle lectures, and engaged in consulting services. His writing has been recognized with numerous awards, and his co-authorship of the textbook, _Design of Concrete Structures_ with T. Y. Lin, made his name familiar to practitioners of prestressed concrete design. The College of Engineering appointed Dr. Burns as Associate Dean for Academic Affairs for the period 1989-93, and subsequently he was appointed Director of Ferguson Structural Engineering Laboratory 1994-97. Upon retirement in 1997, he was named Professor Emeritus for The University of Texas at Austin in 1998.

**Professional Society Leadership and Recognition**

Ned Burns has been a leader in every professional society in which he held membership. He served the Austin Branch of the American Society of Civil Engineers after his return to Austin, as he chaired the Branch Associate Members Activities Committee. He participated in numerous technical committees of ASCE, and he has been a member of the ASCE Technical Publications Review Board throughout his career. With the Accreditation Board for Engineering and Technology, he has served as the Civil Engineering examiner for several review teams. A member of the ASCE/ACI Joint Committee for Prestressed Concrete since 1963, and Chairman 1983 to 1989, he chaired the Ad Hoc Committee that produced _Tentative Recommendations for Prestressed Flat Plates_ published in 1974.

Several American Concrete Institute Committees, Committee 115 on Current Research, as well as the Composite Construction Committee and the Prestressed Concrete Committee 423 benefited from his membership. He was elected to the ACI Board of Directors for a 3-year term 1983-86, and served thereafter as chair for the ACI Publications Committee 1987-92.
Committees of the Precast/Prestressed Concrete Institute have been served by Dr. Burns, including Committees on Prestressing Steel, Education, Segmental Construction, Bridges and Technical Publications. He has been a member of the Board of Directors and Technical Advisory Committee for the Post-Tensioning Institute since 1976.

The patient and considerate actions of Dr. Burns as an advisor continued to be recognized by student groups as he was the recipient of the Ervin S. Perry Student Appreciation Award in 1971, Engineering Student Council Teaching Achievement Award in 1977, Distinguished Advisor Award in 1978, Halliburton Education Foundation Award of Excellence in 1983, the University-wide Amoco Teaching Excellence Award in 1983 and Meritorious Service Award for Graduate Advising in 1986. The college of Engineering designated him Zarrow Centennial Professor in 1983.

The Travis Chapter and the Texas Society of Professional Engineers designated Dr. Burns as their Engineer of the Year in 1987. In recognition of his outstanding record in student teaching and advising, he was awarded the Blunk Memorial Professorship for 1996-97.

Technical Societies have recognized Dr. Burns with specific awards. The Joe Kelly Award in 1990 and a Structural Research Award in 2005 from the American Concrete Institute, the Martin F. Korn Award from the Precast/Prestressed Concrete Institute in 1993, the T. Y. Lin Award from the American Society of Civil Engineers in 1994, and the Distinguished Professor Award of the Prestressed Concrete Institute in 2000. Dr. Burns was inducted into the National Academy of Engineering in February 2000.

Closing Remarks

On behalf of the larger community of educators and practitioners in structural engineering, I express gratitude to the American Concrete Institute for organizing this symposium as a tribute to Ned Burns. This Symposium collection of technical papers by a few authors, students and associates of Dr. Burns is a fitting tribute to the gentleman whom we regard as scholar, teacher and friend. The community of structural engineers and the teaching and practice of structural engineering have been made significantly better as Ned Burns has walked, thought and worked among us.
Ned and Martha honored their parents by endowing the Burns-Fontaine Graduate Fellowship in Engineering
Fig. 3 – Teenage Ned, looks happier than a candidate for academia

Fig. 4 – Ned Burns generation. Ned (far right) with brothers, sisters, and cousins
Fig. 5a – Pvt. N. H. Burns at Fort Belvoir (May 1955)

Fig. 5b – Pvt. N. H. Burns at Fort Ord, CA (March 1955)
Fig. 6 – June 11, 1955
Martha Fontaine became Mrs. Ned Burns
Fig. 7 – Pondering Ph.D. Problems at University of Illinois 1960

Fig. 8 – Dr. Burns studying the underside of a slab after a test
Fig. 9a & 9b – Discussion of a slab test in 1974 with R. Hemakom and C. Chanwatchai
Fig. 10 – Graduate students with the Burns family in 1968 (Kathryn, Ned, Mr. Chintrakarn, Stephanie, Martha, and Mr. Chang)
Fig. 11 – Dr. Burns accepts the first of many awards for teaching excellence

Fig. 12 – Ned and Martha entertain shortly after moving into their new home 1967
Fig. 13 – Ned Hamilton Burns 2005 – Scholar, Educator and Engineer