THE HISTORY OF PRESTRESSED CONCRETE: 1888 TO 1963

by

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Abstract

The concept of prestressed concrete appeared in 1888 when P.H. Jackson was granted the first patent in the United States for prestressed concrete design. Jackson's idea was perfect, but the technology of high strength steel that exhibited low relaxation characteristics was not yet available. It was not until Eugene Freyssinet defined the need for these materials that prestressed concrete could be used as a structural building material. Unfortunately, although Freyssinet, a brilliant structural designer and bridge builder, lacked the teaching qualities necessary to communicate his ideas to other engineers. It would take Gustave Magnel to write the first book of design in prestressed concrete, communicating this idea to designers worldwide. Magnel designed and built the legendary Walnut Lane Bridge in Philadelphia, which revolutionized prestressed concrete in America. Simultaneously, Urlich Finsterwalder, the German bridge builder and designer, was revolutionizing the construction means and methods for prestressed concrete bridges. For example, Finsterwalder invented the free-cantilever construction method of prestressed concrete bridges, which allowed long span bridges to be constructed without stabilized shoring. He then designed stress-ribbon bridges, which would eventually allow prestressed concrete to span distances only steel suspension bridges could achieve. However, it wasn't until Paul Abeles and his peer, H. von Emperger studied and tested prestressed concrete that the idea of "partial prestressing" emerged. Initially, Freyssinet and Magnel were adamant that prestressed concrete should not be allowed to exhibit any tensile forces at sustained loading. Later, the Roebling family developed the first stress--relieved wire followed by the first stress--relieved strand. T.Y. Lin once again brought prestressed concrete back into the spotlight when he organized the First Prestressed Concrete World Conference in 1957. Shortly after this conference, Lin published a technical paper in the Prestressed Concrete Institute (PCI) Journal that introduced a new Load Balancing technique which allowed most structural engineers to design prestressed concrete very easily.

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Acknowledgements

I would like to acknowledge Ken Bondy, PE, SE, for his help and insight on this report. He provided me with very professional knowledge and insight which tremendously helped me writing this report.

Dedication

This report is dedicated to my Dad, Robert L. Dinges, who has inspired me to pursue a career in structural engineering.

CHAPTER 1 - Introduction

During the 1800's, building materials available to structural engineers and builders consisted of cast iron, masonry, timber, and reinforced and unreinforced concrete. Mild steel, structural steel, was developed by Henry Bessemer in 1858 (see figure 1.1 for a flowchart). In the late 1800's, structural steel took the place of cast iron in many structures due, mainly, to its ductile characteristics. Since cast iron is a very brittle building material, very little visual stress in the form of deformations were visible before material failure occurred. However, new ductile structural steel exhibits significant deformations before brittle failure occurs. Consequently, many bridges of this era were constructed of steel or cast iron, especially long span structures. In many other places including isolated areas and areas prone to much corrosion, reinforced concrete or masonry was the choice for the building material. Meanwhile, masonry and reinforced concrete bridges relied on arch construction to maintain their structural integrity.

One major problem with masonry arches was the keystone sagged for some reason. This produced very unfavorable deflections at mid-span of bridges. This sag at mid-span determined the limiting span length: the longer the span, the more deflection occurred. This did not make sense; most builders placed all masonry in the arch except for the keystone on formwork. After the masonry had cured and all theoretical shrinkage had occurred, the keystone was placed, and the arch was in place. This method only allowed for a small amount of shrinkage to take place in the keystone joints. A brick masonry railroad arch bridge is depicted in Figure 2-32 later on in this report.

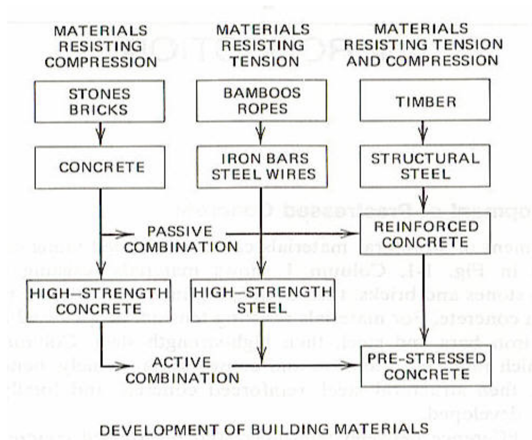

Fig. 1-1. Development of building materials.

Figure 1-1.1 Development of Building Materials (Kramer 2005)

To correct this problem, an external force had to be exerted on the structure after it was placed and cured. After that force was exerted, the keystone was once again reset and allowed to cure. This idea of applying an external force on a structure after-the-fact would prove to be the foundation for the founders of prestressed and post-tensioned concrete. Indeed, prestressed concrete can be defined as ordinary concrete that has a compressive force enacted on it by means of an external force, usually applied by tensioned internal high-strength steel cables or tendons. In prestressed concrete, the cables are tensioned before the concrete is poured, whereas in post-tensioned concrete, the cables are tensioned after the concrete is poured and cured around the cables.

The difference between prestressed concrete and ordinary reinforced concrete is: the reinforcing steel in reinforced concrete is placed in the concrete to resist flexural stresses applied to the member by applied loads. The concrete resists the loads to a certain point, after which it cracks and the reinforcing steel is engaged. Once the steel is engaged in resisting tensile forces, the concrete no longer does.

This idea worked fairly well with masonry, but coupled with a building material such as concrete, the possibilities would turn out to almost be endless. Prestressed concrete, much like reinforced concrete earlier in history, brought concrete, steel, and masonry together for a very versatile building material, which, in many designers' minds, could produce a competitive alternative for design with any material. One designer went as far as developing a method of prestressing to compete with modern steel suspension bridges in span as well as structure depth. Clearly, once prestressed concrete entered structural engineering, designers expanded its uses for most structural engineering applications.

CHAPTER 2 - The Beginnings

At the beginning of the twentieth century,*"Prestressed Concrete"* soon became the single most significant new direction in structural engineering according to Billington (2004). This unique concept gave the engineer the ability to control the actual structural behavior while forcing him or her to dive more deeply into the construction process of the structural material. It gave architects as well as engineers a new realm of reinforced concrete design pushing not only the structural but also the architectural limits of concrete design to a level that neither concrete nor structural steel could achieve. Ordinary reinforced concrete could not achieve the same limits because the new long spans that prestressed concrete were able to achieve could not be reached with reinforced concrete. Those longer spans required much deeper members, which quickly made reinforced concrete uneconomical. Additionally, steel structures weren't able to create the same architectural forms that the new prestressed concrete could.

Prestressed concrete was an idea of structural designers since P.H. Jackson of the United States (U.S.) patented his idea in 1888 (Refer to Appendix 1 for P.H. Jackson's patent.) as a method of prestressed construction in concrete pavement. The reason prestressed concrete was not used as a building material in the early years was the lack of technology to support the idea. For example, metallurgists had not yet discovered high strength steel, which combined the needed high compressive forces in a minimal amount of steel with low relaxation characteristics that minimized creep and post-stress deformations in the prestressing steel; therefore, the idea hibernated until Freyssinet reexamined it in the early twentieth century, the first to actively promote prestressed concrete.

2.1 Eugene Freyssinet

Growing up, Freyssinet was not an engineering genius. In fact, Billington (2004) points out that Reyssinet was a mediocre student, rejected by École Polytechnique (the "Polytechnic School"). The École Polytechnique, L' École, often referred to by its nickname X, is the foremost French grande école of engineering according to French and international rankings. Founded in 1794, it is one of the oldest and most prestigious engineering schools in the world, with a very selective entrance exam. As one of the world's foremost establishment in science, the École Polytechnique trains graduates who become outstanding scientists, researchers, and managers. The École Polytechnique ranks among the best universities of the world, even among the top three according to Professional Ranking of World Universities 2007. However, Freyssinet was accepted the following year "with the not very brilliant position of 161st". He then went on to graduate 19th and succeeded upon graduation at being accepted to the École des Ponts et Chaussées and the world's oldest civil engineering school, and one of the most prestigious French Grandes Écoles of engineering. There, for the first time Freyssinet's "artisan love of building coincided with that of his teachers, and it was there, in the lectures of Charles Rabut in 1903-04, the idea of prestressing first came to him."

"The idea of replacing the elastic forces that are created in the reinforcements of concrete by deflection due to loads, by previously imposed and permanent stresses of sufficient value, came to my mind for the first time during a series of lectures given by Charles Rabut at the École des Ponts et Chaussees in 1903-04. These lectures were devoted, on the one hand, to reinforced concrete and, on the other hand, to the systematic study of spontaneous or provoked deflection in structures. (Billington 2004)"

Upon graduation, Freyssinet became a bridge builder in the wilderness of south-central France. In doing so, he accidentally learned of another phenomenon not then defined in structural engineering. Billington quotes Freyssinet's words, "Towards 1906-07, the idea of applying pre-compressions was firm enough in my mind to lead me to draw up a project for a 2500-ton capacity tie linking the two abutments of a 50-m span trial arch (2004)."

According to the Department of Civil Engineering, Civil Engineering Structures Group at the University of Cambridge (2004), Freyssinet said that one day, a few months after the project was completed, he was cycling to work over this bridge, and he realized that the parapet was no longer straight; instead it was dipping at the mid-span of the arch. From his observations, he concluded that the arch must have shortened, which allowed this dipping action. Luckily, he was able to reinstall the arch jacks and fix the structure. This led him to realize that concrete slowly deforms under load over time. He also recognized that this deformation is permanent, and when loads were taken off the structure it did not go back to its original position. Freyssinet later realized that he had just discovered the phenomenon of *creep* in concrete structures. Creep is defined as the tendency of a material, specifically concrete, to permanently deform slowly over time while under the influence of stresses. He performed tests to confirm this and concluded that the early attempts at prestressing had failed because concrete of too poor quality and/or low strength had been used. He also concluded that steel bars with too little prestress force had been used as well. This means that the amount of creep was heightened by the low strength concrete, which caused the creep strains to elevate removing the prestress force. Freyssinet explains his discovery of creep, "This tie and its arch were completed during the summer of 1908, but, a study of their deflection and other observations taught me the existence of creep in concrete, a phenomenon that was then unknown and even energetically denied by official science."

The bridges over the Allier River were another example of his great engineering and construction. He volunteered to build, for one-third the price that had been bid, all three bridges. He proposed to the highway department, in return that all other bids be rejected and he be allowed to act as project engineer and as the builder of his designs. Consequently he was given complete control of this project. Several months after completion, the 238 ft span arches began to deflect downward at an accelerating rate. To correct this deflection, Freyssinet removed the joints at the arch crown and jacked the joint with Freyssinet's flat jacks. This negated the increases of stress resulting from the deformation of the neutral axis of the arches--the first time a post-tensioning application was used in reinforced concrete. This example did not use strands placed in ducts within the concrete, typically thought of today as post-tensioned concrete, but it did negate the effects of stresses caused by concrete dead load by applying an external force.

Throughout these processes, Freyssinet laid the groundwork for prestressing; however, almost 20 more years would pass before high strength steel would be utilized for anything other than a special method of arch construction. During the 1920's, Freyssinet designed two world-record breaking arch bridges with spans of 432 ft (131.7m) and 614 ft (187m). Once again, these arched bridges were jacked apart at their crowns by a controlled prestress. Billington commented on Freyssinet's accomplishment with the following statement: "Had he never pursued the idea of prestressing, he would still have been regarded, along with Robert Maillart, as one of the two greatest concrete structural engineers in the first half of the twentieth century. (2004)" Freyssinet was without a doubt one of the masters of long span, reinforced concrete bridge design.

In 1928, Freyssinet recognized how significant prestressing was, and he patented his ideas, devoting the next four years to developing the potentials of prestressing. His patent involved high strength steel wires tensioned in concrete beams. This was the first time that prestressing steel was used in a concrete member to counteract tensile forces, thereby substantially reducing the amount of flexural reinforcing steel.

In 1932, the editor of a new journal, *Science et Industrie*, asked Freyssinet to write about his progress in prestressing as well as other tests and their results in an article titled "New Ideas and Methods." Eventually in his fourth of six chapters, he outlined the "conditions for practical use of prestressing." Billington (2004) states the conditions as follows:

- Using metals with a very high elastic limit.
- Submitting the steel to very strong initial tensions, much greater than 70,000 psi.
- Associating the metals with concretes of a very low, constant and well--known rate of deformability, which offer the additional advantage of very high and regular strengths of resistance."

Additionally, Freyssinet recognized the need for the following material qualities:

- high strength steel, greater than 70 ksi
- high initial stress of tensioned steel
- high strength concrete to reduce to a minimum the loss of initial prestress

This type of steel was needed so very high tensile forces could be induced in a relatively small cross-sectional area of steel, and also to override the effects of creep and shrinkage in steel. A ten ksi (69 mPa) decrease in the strength of steel due to the effects of creep and shrinkage affects 20 ksi (138 mPa) steel much more than 200 ksi (1380 mPa) steel in percent decrease of strength. High strength steel also greatly reduced the area of steel needed, and when stretched, retains the induced tensile stress and could transfer these forces from the steel to the concrete without much loss. Finally, to transfer these forces to the concrete, high strength concrete had to be developed to avoid crushing.

This was the first time that an engineer had based the idea of prestressing on a clear understanding of the properties of concrete and steel. However, the problem with Freyssinet's prestressed concrete was finding any commercial value for it at the time. Also, some of the aforementioned materials, high strength steel and concrete, and material qualities, low relaxation steel, had not been invented yet. This problem was compounded by the fact that France was affected by the worldwide economic depression; in times of economic crisis, very few people are willing to invest in a new business venture. If the construction market had been strong and bidding for jobs had been competitive, a demand for new ideas and construction techniques likely would have been present. These new ideas possibly would have lowered overall building/bridge costs and expanded architectural parameters such as maximum clear span as well as floor thickness and story height. In a time of economic prosperity, Freyssinet's idea of prestressed concrete may have taken off immediately.

Eventually, in the early 1930's, Freyssinet opened the first prestressing factory at Montargis, France, where he manufactured prestressed concrete poles for telegraph lines. He used thin concrete tubes made with mortar and prestressed with piano wire (Department of Civil Engineering 2004). Figures 2-1 and 2-2 show the molds for the poles and the placing of the poles at Montargis, respectively. Due to the economic depression, the factory was without business and not long after his article was published in the New Journal, his factory went bankrupt.

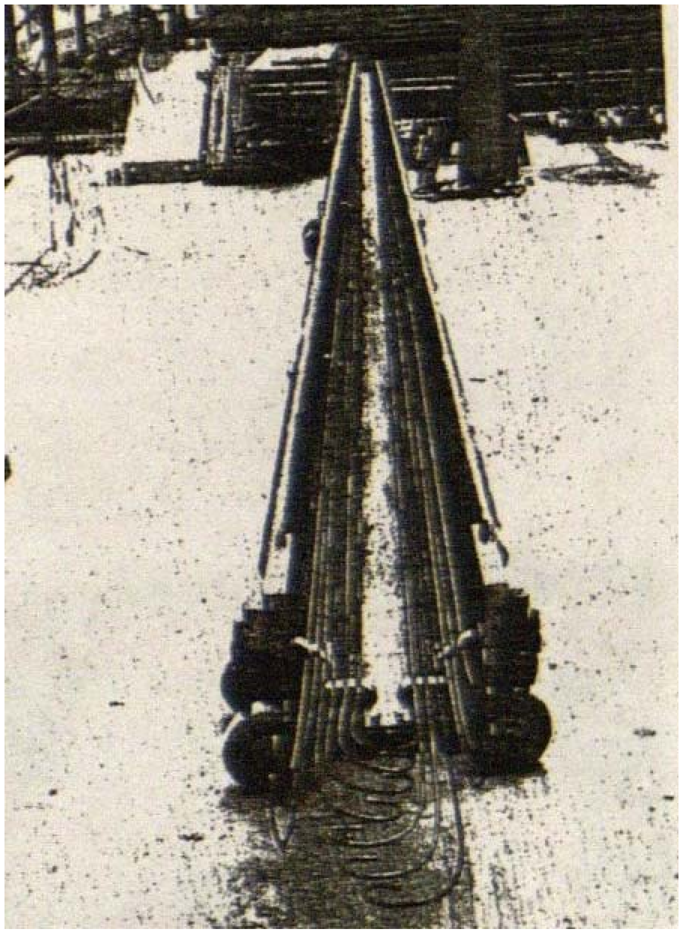

Figure 2-1: A half-mold containing steel reinforcements tensioned for Montarig poles (Freyssinet 1932)

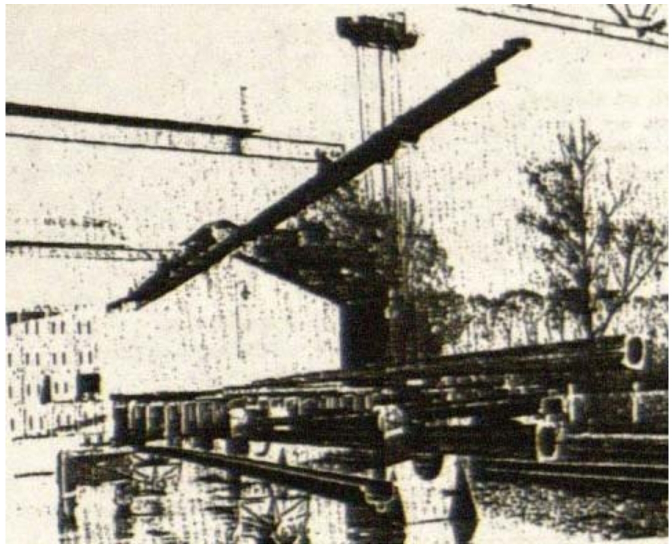

Figure 2-2: 12m poles of 25mm (1") thickness (Freyssinet 1932)

However, his factory was not closed for long. In 1935, the Maritime Terminal at Le Havre was settling into the Harbor at a rate of about one inch per month. Freyssinet successfully stopped this settlement by consolidating the foundations by prestressing. This effort helped Freyssinet gain numerous large scale projects from the French authorities in the years to come.

Freyssinet realized to be successful as a prestressing manufacturer; he had to develop a practical system to prestress high strength steel in concrete members. His main concern was to develop an anchorage for the prestressing steel to avoid slip after initial prestressing. If the prestressed wire slips in a concrete beam after it is cast, most of the prestressed force will be lost due to shortening in the wires. Furthermore, if the prestress force is lost after the beam is cast and no flexural steel is in place, the beam will fail long before the ultimate theoretical load is placed. The only resisting force in place is the tensile strength of the concrete, which is very low. Consequently he used two larger diameter wires (typically 5 or 6mm, 0.196"or 0.236") clamped by means of a single wedge between the wires pushing them against an external block. Once he developed this method, he immediately patented it in France (Department… 2004). A description of his work was given by Gueritte in 1936: "A development of the original anchorage is this system, which can grip 12 wires of 5 mm diameter. The central wedge is grooved to hold the wires and is made of high-strength mortar but with external and internal spirals of steel. The barrel is cast into the structure and connected to the duct for the tendon. After the concrete has hardened, the 12 wire strand is inserted and jacked, using the wedge to grip the tendon (Department…2004)." This is shown in Figure 2-9 later in section 2.2.1.7 "Methods of Prestressing."

Eugene Freyssinet established the practical use of prestressed concrete in structural design and construction and the parameters that made prestressed concrete possible in engineering applications. As Billington stated, his passion for prestressing went on to define prestressed concrete as an entirely new material with the widest possible application. Ultimately, Freyssinet considered reinforced concrete and prestressed concrete as two completely different building materials. He believed that a structure is either fully prestressed, or it is not to be called prestressed at all.

Freyssinet knew the concept and method of prestressed concrete thoroughly, as displayed through his brilliant bridge designs and his patented anchorage devices. He clearly could communicate his passion for prestressing through design and construction, but he could not put in writing his technical concepts (Billington 2004).

It would take another individual, Gustave Magnel, to more clearly communicate the technical aspects of prestressing to others and eventually, to the United States.

2.2 Gustave Magnel

Gustave Magnel graduated from the University of Ghent in Belgium with his degree in Civil Engineering in 1912 (Taerwe 2005). He then spent the years of World War 1 (WWI) in London, England, employed by a London contractor from 1914 to 1919. He helped British engineers learn the design and construction of reinforced concrete. It was during this period that Magnel developed his extraordinary teaching talent. This also gave him the opportunity to learn to speak fluently in English.

In 1922, Magnel was appointed lecturer in reinforced concrete at the University of Ghent (Department…2004). With Magnel's experience in reinforced concrete construction and design, and then teaching, he realized that to further develop this structural system, he needed to conduct research, for which he needed a laboratory. Magnel had to go through many political and financial difficulties before convincing University of Ghent administrators that he needed a laboratory, but finally after battling for almost five years, he got one.

He was named professor and director of the Laboratory for Reinforced Concrete in 1927 at the University of Ghent in Ghent, Belgium (Billington 20). This lab was located in the basement of a former hotel, and it contained a 300 kN (67,500 lb) universal testing machine and a 3000 kN (675,500 lb) compression testing machine (Taerwe 2005), both of which he could use at his discretion. Magnel wrote about his effort to open this laboratory and keep it functional during the late 1920's:

"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 cannot happen in 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 continues, "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…" (Taerwe 2005). At this time in history, he was considered very bold statement for saying that it is up to the professors to persuade university management that their endeavors are worth investing money in. If the professors failed to do this, many times, the new technology would have failed, or would not have been allowed to develop because of lack of funds. He goes on to say that with even further initiative, professors can keep their labs open with moneys that they make performing tasks in that lab. Looking back, this statement holds true for many of the most high tech laboratories around the country. Many times, a given professor has kept the lab functional with funding from outside sources.

Magnel was originally of French descent, but learned Flemish when in the late 1920's, Ghent changed languages to Flemish (Dutch). This allowed the brilliant Magnel to teach fluently in three different languages, including English. Magnel was such an effective and interesting lecturer that many of his students claimed to have attended the same lectures in two or three different languages

Magnel had luckily developed these skills by the time World War II (WWII) began and isolated him to Belgium. In fact, the Germans did not allow Magnel to teach during WWII, but did permit him to continue to be the director of his laboratory. It was during this isolation that Magnel was able to explore Freyssinet's ideas on prestressed concrete and his own research and testing on prestressed concrete. Subsequently, Magnel carried out full--scale research on prestressed concrete girders. He was also able to study Freyssinet's discovery of creep in more detail with the technology in his laboratory continuously monitoring the effects of loads on prestressed concrete elements. He mainly investigated creep of high-strength wires and creep and shrinkage of normal reinforced concrete. This helped him see that high-strength wires used as prestress wires creep much less than low-strength mild reinforcing steel used in ordinary reinforced concrete. Presumably, Freyssinet proposed that loss of prestress due to creep only existed in concrete because he hadn't run tests that proved that creep also existed in steel. Magnel found through testing that Freyssinet missed a large contributor to creep; in fact prestressed wires were a more significant contributor to creep in prestressed concrete structures than the concrete itself. Considering both the creep of the steel and the creep of the concrete, the loss of prestress that Magnel found was almost double that of Freyssinet's determinations.

During WWII, the German's forbade Magnel to have contact with the French, making it impossible for him to obtain Freyssinet's system of prestressing concrete. Therefore, Magnel promptly developed his own post-tensioning system, the *Magnel-Blaton* system.

This system, discussed later in this chapter, allowed him to perform advanced testing on prestressed concrete. Taerwe explains the *Magnel-Blaton* system, which is seen in Figure 2-10, as follows:

"The anchorages of this system consist of several '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 plate [by] which two wires are secured with a steel wedge. In this way, the stress in the different wires of one tendon is more uniform than in the case [where] all the wires [are] stressed at once. Moreover, a fairly small jack could be used for stressing the wires. 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 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 tendons. Due to this arrangement there was a free space around each wire, which allowed a grout cover by the injection grout, which is essential for protection against corrosion (Taerwe 2004)."

The *Magnel-Blaton* system continued to be used in Belgium until the 1960's. By this time, newer, less labor intensive methods were discovered requiring a different anchorage to hold the higher strength steel tendons.

WWII ended on August 15, 1945, and building in Europe began at a very fast pace. The rebuilding of the infrastructure, which had been destroyed, was the main focus in this period in Europe. By this point, the internal combustion engine had long been invented, and the use of motor vehicles and tanks was common practice for both the Allied and Axis powers. The main targets for the Allies had been key bridges with high Axis use. If the Allies could limit Axis mobilization and isolate enemies to one area, they had a large advantage. As a result of this, many of the major bridges and roadways had been destroyed throughout Europe. To be effective in times of war, a country's defense system needs to be mobile. Therefore, bridge and infrastructure construction had been abundant.

At this time, Magnel was one of the few engineers with extensive experience in reinforced concrete design and construction who had mastered the ideas of prestressing, and had the ability to communicate these ideas to the English-speaking world essentially the United States and Europe as English started to become the dominant language. Countries in Europe started teaching English in schools to help give the young and upper hand. At this point, everyone in the world wanted the opportunity to go to the U.S, and the first step in doing this was learning the language. The United States was beginning to be the world power at this point in history. The period from the end of WWII to the early 1970's was considered as the golden age of Capitalism in America. Magnel, having already learned English, already had an upper hand on most European engineers. He was able to communicate with people throughout Europe to help in the rebuilding after WWII.

In 1948, Magnel wrote his tenth book entitled *Le Béton Précontraint* (Prestressed Concrete), which was immediately published in English. It went through three editions in Britain and then was later published in the United States (Billington 2004). Magnel had an uncanny ability to write successful books, but even more esteemed was his ability to convey his thoughts and ideas in a classroom. In his article, Billington (2004) states, "As one of the few Americans who followed a complete sequence of his courses at Ghent, I can state unequivocally that he was the best teacher I ever had." One of his main goals in teaching, writing, and research was to simplify very complex mathematical and theoretical problems. As stated in *Le Béton Précontraint*:

"In the writer's opinion this problem (of computing the ultimate strength of prestressed beams) should be solved with the least possible calculations, as calculations are based on assumptions which may lead to wrong results…It is therefore proposed to use known experimental results to produce a reasonable formula, avoiding the temptations to confuse the problem with pseudo-scientific frills." Magnel thought that many scientists included unnecessary material in their books just to confuse people whom they considered less intelligent in the subject than they, and those scientists enjoyed doing this.

Figure 2-3 Gustave Magnel (University of Ghent 2009)

2.2.1 ***Le Béton Précontraint*** (Prestressed Concrete) Third Edition" by Gustave Magnel

In the preface of his first edition of *Le Béton Précontraint* Magnel gives credit to Eugene Freyssinet, "Monsieur Freyssinet pointed out that a permanent compressive stress in concrete can only be maintained with steel having a very high yield point or yield stress" Freyssinet was the first to accept this truth and to make prestress concrete practical. It is true creep and shrinkage can cause a 10 to 20 percent loss of the initial stress, but this is admissible if allowed for in the calculations (Magnel 1954)."

2.2.1.1 The Principle of Prestressed Concrete

Magnel explains the weaknesses of concrete as a structural element, and therefore a contributing factor in the development of prestressed concrete. If concrete was just as strong in tension as it was in compression, reinforcing steel would not be needed. If reinforced concrete did not shrink in the curing process or creep from loads over time, the demand for something better would not have occurred. In addition to strength, crack control was a major issue for architects as well as the general public. From a structural standpoint, other than an issue with corrosion, cracks are needed to engage the reinforcing steel, and therefore are not a hindrance to developing the strength of concrete. Thus, Gustave Magnel begins his book by explaining the need for prestressed concrete.

Concrete is a weak building material for three main reasons. The first reason is its material limitations; achieving a compressive strength in concrete equal to 6,000 pounds per square inch (psi), 41,370 kPa, is fairly easy, but reaching tensile strengths of 1,000 psi (6895 kPa) in concrete itself is almost impossible. Concrete tensile strengths are approximately 1/8 to 1/10th of the 28-day compressive strengths. This is a huge problem when concrete is used as a flexural member. A simply supported concrete beam loaded from the top, for example, has one-half of its fibers in compression and one-half of its fibers in tension. However, concrete will fail in tension due to cracks propagating from the bottom center of the beam, resulting in a brittle failure, the least favorable sort of failure because it happens quickly and without warning. This flaw necessitated placing reinforcing steel, which has high tensile strength, in the tension regions of concrete beams, which is depicted in Figure 2-4. Joseph Monier developed this idea of reinforced concrete and received a patent for it in 1849. Once concrete cracks in the tensile region, the reinforcing steel, with proper bond, engages and resists the tensile forces. At this point, the concrete in the tension region can be discounted since it provides very little tensile capacity. Designed with the steel yielding prior to the concrete crushing, reinforced concrete fails in a ductile manner, which is much more favorable than a brittle failure. However, if too much reinforcing steel is installed in a concrete beam and the beam is loaded to the point that the steel does not yield, the concrete in the compression region may crush.

Another main problem with concrete is that to be workable, it requires more water than is required to hydrate the cement causing the chemical reaction. This means the required strength is forfeited with the presence of more water, i.e. higher water—to—cement ratio. Once the cement has hydrated, the excess water evaporates from the concrete leaving voids, which cause shrinkage cracks. In addition to these shrinkage cracks, reinforced concrete must crack before the steel is engaged. However, in outdoor applications such as bridges, these cracks cause the reinforcing steel to corrode due to exposure to water.

When steel corrodes, it expands in volume. This change in volume exerts stresses on the surrounding concrete causing the concrete to crack further. This additional cracking allows rain water to penetrate the concrete, and in colder climates this trapped water freezes and exerts stresses on the surrounding concrete. This additional stress causes the concrete to crack further and the reinforcing steel to corrode more. This cycle repeats itself, possibly several times a season depending on the location, causing loss of bond between the reinforcing steel and the concrete and reduced section of the reinforcing steel. This decreases the capacity of the reinforced concrete section until the section can no longer resist the external applied loads for which it was designed. Finally, reinforced concrete performs well when properly designed; many architects believe that cracked concrete is not aesthetically pleasing.

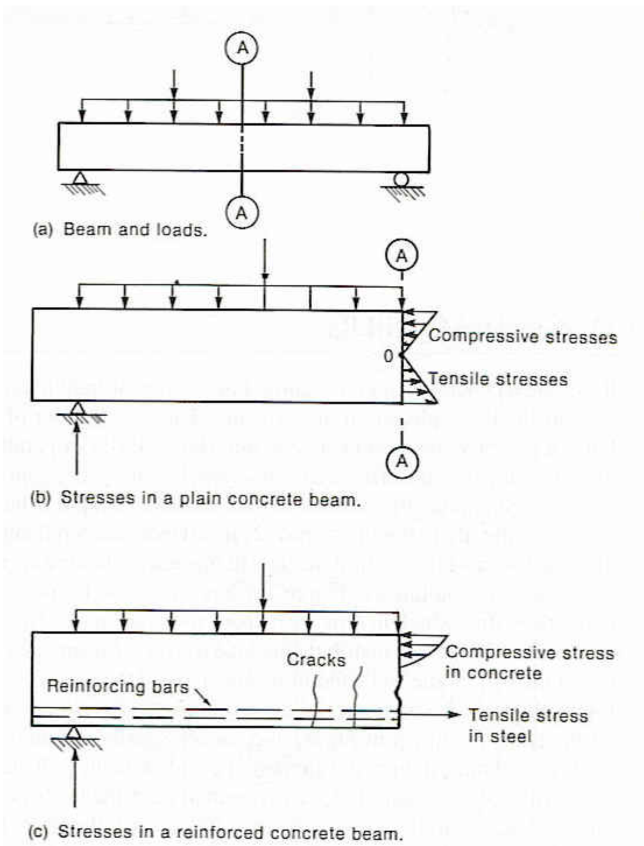

Figure 2-4 Theory of Reinforced Concrete in Flexure (Kramer 2005)

Magnel's second reason that concrete is a poor building material is the effects of diagonal tension, shearing stresses, often requires unfavorable beam depths. At this point, engineers didn't fully understand diagonal tension in concrete beams, so instead of adding stirrups to resist these stresses, they increased the beam depth. Unfortunately, a large beam that spans a great distance means a very high dead load due to the concrete weight given that the compressive strength of concrete is roughly 5 to 10 percent that of steel while its unit weight is roughly 30 percent that of steel. A concrete structure requires a larger volume and a greater weight of material than does a comparable steel structure. For bridges, this becomes impractical very quickly.

The third reason Magnel says reinforced concrete is a poor building material is that the full potential of high strength concrete, compressive strength greater than 6000 psi, cannot be achieved with mild steel because the concrete will crush. If the size of the beam were reduced to take full advantage of the compressive strength of high strength concrete, the amount of reinforcing steel needed to resist high tensile forces would make the beam uneconomical. More simply stated, it would be impossible to fit the amount of steel needed to resist tensile forces in the area of the beam, which would have been reduced in size due to high strength concrete. At first glace, a simple fix to this problem would be to increase the yield stress in steel, making it one sixth that of ordinary mild steel reinforcing. This would allow the steel area to be reduced by one sixth, therefore making a once uneconomical beam, economical. However, this solution was unacceptable because the strain of high yield stress steel is about six times that of mild steel. If the stress and strain are proportional, the amount of stress applied directly affects the strain in the beam, causing deformations in the steel that are transferred to the concrete member as cracks. This creates wider cracks in the concrete than does mild steel, making crack control more difficult.

Simply stated, "Prestressed concrete is a remedy for these weaknesses (Magnel 1954).":

"Let us assume that we succeed in applying to a prism of plain concrete a uniform pressure in all directions of, say, 2000 lb. per square inch (7790 kPa). If this prism were placed on two supports and forces caused to act on it, it would not crack as long as the load alone did not create tensile stresses higher than 2000 psi (13.8 mPa) plus the tensile strength of the concrete. This, briefly, is the principle of prestressed concrete: the compression induced by the external pressure applied to the beam is the "prestress". (Magnel 1954)"

This explanation means if an engineer induces a tensile stress to concrete greater than the stress from the external loads, the concrete will not only resist the loads, but will also resist cracking.

The concept of prestressing appears in everyday life. To demonstrate the idea of prestressing, Magnel puts numbers to this model:

"Assume that ten books make up a beam six inches wide, ten inches high, and twenty inches long, and that each book weighs two pounds. The bending moment would then be 20*20/8 = 50 in.-lb. and the tensile stress would be (50*6) / (6*10^2) = 0.5 psi. The absence of tensile strength prevents the development of the tensile stress.

Assume now that we compress, longitudinally and without eccentricity, the beam of books with our hands with a force of 36 lb., resulting in a stress of 36/(10*6) = 0.6 psi. Under this condition the books form a beam capable of carrying its own weight on a span of 20 inches; the beam has a compressive stress in its top fiber of 0.6*0.5 = 1.1 psi and in its bottom fiber of 0.6-0.5 = 0.1 psi. There is compression on the whole cross section, and the beam is completely same."

After experimenting with different eccentricities and the force that has to be exerted at these eccentricities, engineers could quickly see that if this force were lowered to the lower third of the beam, much less force would be required to keep the beam intact. Therefore, external tensile stresses are required only in the region of beams that have opposing internal tensile stresses acting on them.

To render a greater understanding of the benefits of prestressed concrete, the design of a mild reinforced concrete slab is compared to that of a prestressed concrete slab with the same general conditions and loading. One important variable to be considered is the weight of concrete. If prestressed concrete can greatly reduce the depth of the slab, the prestressed slab will have a much smaller dead load due to lower self weight. It was very important for Magnel to establish the benefits of prestressing early in his book, and he would do it by providing this comparison. The following example is most likely the first short but detailed description and comparison between the detailing, size and amount of mild steel reinforcing needed in a reinforced concrete slab versus a prestressed slab.

2.2.1.2 Comparison of Mild Reinforced Concrete to Prestressed Concrete…

The following outlines Magnel's comparison of two bridge slabs with the same spans and loads except for the differing self-weights.

The design of a bridge slab will be considered with a span of 66 feet (20m). This bridge slab will have to carry its own weight as well as a superimposed load of 400 pounds per square foot (psf) (19.15 kPa) (Figure 2-5).

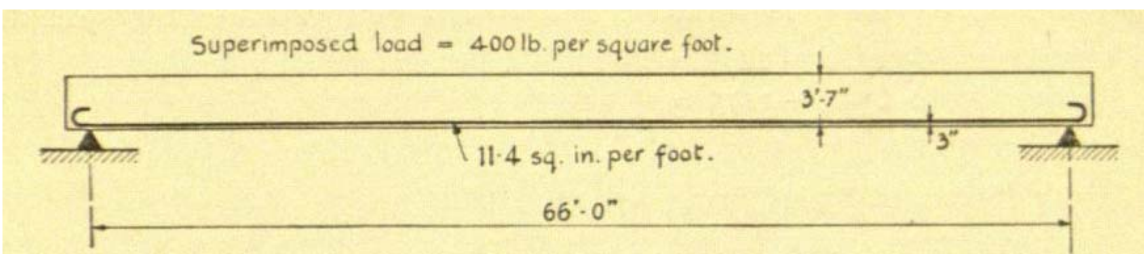

Figure 2-5 Superimposed Load (Magnel 1954)

2.2.1.2.1 Ordinary Reinforced Concrete

The design indicates a 3'-7" (1.1m) thick bridge deck is required with a slab dead load of approximately 540 psf (25.9 kPa). To resist tensile forces from the applied loads, 11.4 square inches per foot (in^2/ft.) (10590 cm^2/m) of mild steel reinforcing placed 3 inches (7.6 centimeters) above the bottom of the slab is required. Due to the 940 psf (45 kPa) loading, the following stresses are present: Compressive stress in the concrete, 1,430 pounds per square inch (psi) (9860 kPa); tensile stress in the steel, 16,600 psi (115mPa); diagonal tension, 80 psi (552 kPa). He states, "This is obviously not a good design," as it suggests very high stressesz for ordinary concrete. The mild steel reinforcing is about 2.4 percent of the gross area of the section. If this reinforcing steel is placed in one layer in the beam it would require 1 ½ inch bars at 1 7/8 inch centers, making this a very impractical design because there would not be enough concrete between the reinforcing steel to produce adequate bond. If the concrete aggregate was greater than 1/8", it would not be able to pass through the steel, leaving voids in the concrete. As quoted in *Le Béton Précontraint*, "This design is used as an example in order to avoid the possible criticism that the ordinary reinforced concrete slab has been made too heavy so that the advantage of prestressed concrete is more apparent (Magnel 1954)."

2.2.1.2.2 Induced Compression

Assume now that this same bridge is to span a trench cut in rock. Also assume that the supports are the rock that this trench is cut from (Fig. 2-6). A 32 inch (81.3 cm) thick concrete slab with no reinforcement is built to span the trench. This slab has to be formed and shored until it reaches full compressive strength or the compressive strength that concrete achieves after curing for 28 days. The slab is built so that the left hand is butted up against the hard rock and the right hand side ends a short distance from the rock. When the concrete reaches 28-day compressive strength, a hydraulic jack is placed in the gap on the right hand side and induces a compression force of 268,000 pound per foot (3911 kN/m) two inches (5.1 cm) above the bottom of the slab. This creates an eccentricity of 14 inches (35.6 cm) from the center of the slab to the jacking force. Due to this force, the bridge deck deflects upwards so it is no longer in contact with the shoring. Once the slab leaves the shoring, the dead load immediately acts. As the force of the jack increases, the compression in the beam due to this force proportionally increases. The dead load of the slab increases as the slab deflects off the shoring thereby increasing the tension due to dead load in the bottom of the slab at the same rate. "Therefore, the action of the jack coincides with the action of the dead load (Magnel 1954)."

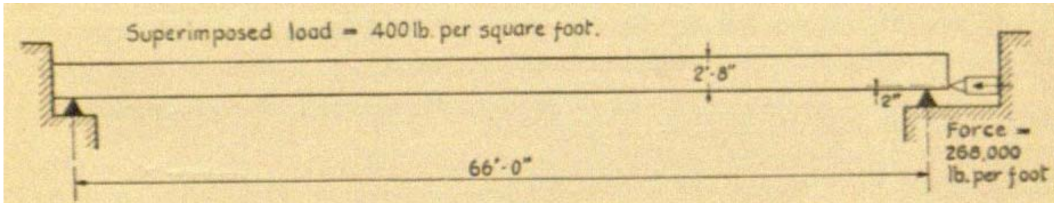

Figure 2-6 Slab spanning solid rock abutments (Magnel 1954)

The following paragraph addresses the stress distribution in the slab at mid-span: first by determining if the 268,000 lb jacking force alone was applied. The stresses calculated from the ordinary formula for eccentrically loaded on homogeneous sections are as follows:

Top Fiber: (268,000/(32*12)) * (1- (6*14/32) = 1120 psi (7722 kPa) (tension) (Eqn. 2-1)

Bottom Fiber: (268,000/(32*12)) * (1+(6*14/32) = 2550 psi (8764 kPa) (compression) (Eqn. 2-2)

The dead load is 400 psf as before, which is the same as the superimposed load, as well as the bending moment due to this dead load, 217,800 ft.-lb (295 kN-m), and the stresses due to these loads, 1,275 psi (4,966 kPa) (tension). Next these stresses are added to the initial prestress stress as shown in Figure 2-7A and Figure 2-7B. These forces act concurrently; therefore, they can be combined as shown in Figure 2-7C.

Top Fiber: -1,120 + 1275 = 155 psi (605 kPa) (compression) (Eqn 2-3)

Bottom Fiber: 2,550 – 1275 = 1,275 psi (4966 kPa) (compression) (Eqn 2-4)

Clearly after the dead load acts on the slab, which acts after the prestress force is in place, the beam has no tensile stresses; therefore, there is no need for mild tensile reinforcement. Next, the addition of the superimposed loads on the slab is examined; this load is equal to the dead load referred to in Figure 2-7D.

Top Fiber: +155 + 1275 = 1430 psi (5570 kPa) (Eqn 2-5)

Bottom Fiber: 1275 – 1275 = 0 psi (0 kPa) (Eqn 2-6)

With a prestress force of 268,000 lb. per foot, mild reinforcing steel is not required since no tensile forces are acting on the slab.

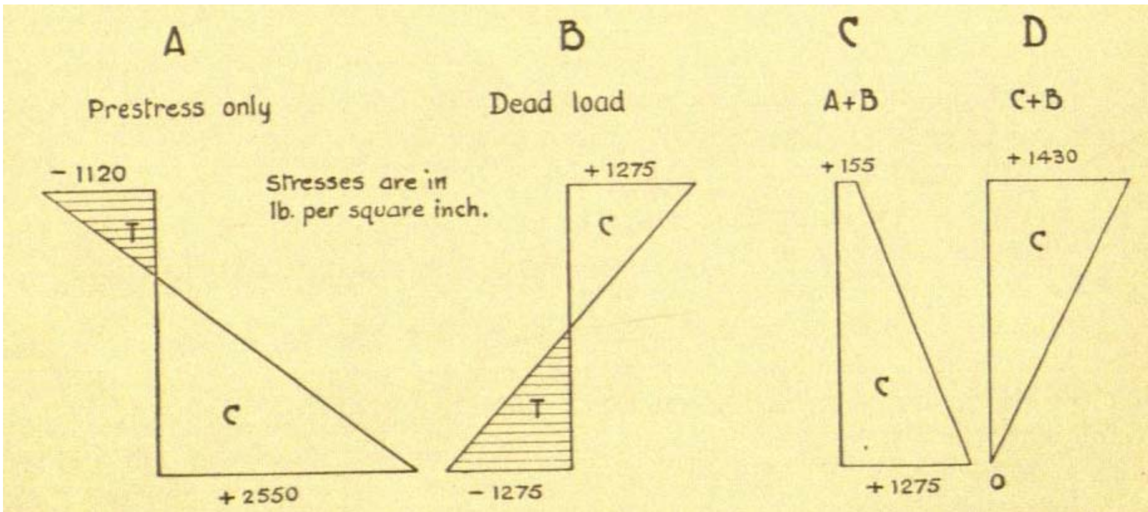

Figure 2-7 Stress distribution through loadings (Magnel 4)

To determine the required thickness of the slab requires considering the vertical shearing force on the beam. Due to the loads imposed, the vertical shearing force is 26,400 lb. per foot of width (385 kN/m) of the beam. This would result in a diagonal tensile stress of 84 psi (327 kPa). (This diagonal tensile stress referred to throughout this example is what we consider shearing force in concrete members today. In designs conforming to the American Concrete Institute Specifications, (ACI), if the shearing stress is greater than 1/2*phi*Vc, stirrups have to be added to the beam (ACI 2005). Phi is a strength reduction factor, 0.75, and Vn is the nominal strength of the concrete in shear. In this design, instead of adding stirrups, the beam depth is increased to add to the value of Vn. However, since this beam is prestressed concrete, neither of these analysis methods is valid. At the neutral axis in this slab, for pure bending, a longitudinal compression of 715 psi (2,785 kPa) exists, which allows for a reduction in diagonal tension. The ordinary theory of elasticity may be used to compute this new diagonal tensile stress. While concrete is not an elastic material, for simplicity of design it can be considered in the elastic range:

$\sqrt{102^2 + (715/2)^2} - 715/2 = 15$ psi (58 kPa) (Eqn 2-7)

2.2.1.2.3 Results of Comparision

The results of this short design example from Magnel are the following: The required thickness of the mild reinforced concrete beam is 3 ft 7 inches (1.09m) while the required thickness of the prestressed concrete slab is 2 foot 8 inches (0.81m) to resist diagonal tension. The mild reinforced concrete beam requires a very large amount of mild steel while the prestressed beam requires no mild reinforcing steel. The maximum concrete stress of 1,430 psi (9,860 kPa) remains the same in each example even though the maximum diagonal tension is lowered from 80 psi (312 kPa) to 15 psi (58 kPa). With the upward deflection created by the prestress force, the shoring of the beam is easily removed. For a design such as this, a solid rock abutment has to be present at each end of concrete beam, and a mechanical jacking device capable of producing 268,000 lb. per foot of prestressing force effectively in the concrete beam must be used.

Magnel knew the importance of the prestressing, and the example he provided explains the concept and application of prestressed concrete. With these major concepts established, however, other parameters need to be defined: justification of the need for high strength steel, the need for a parabolic curvature in prestress cables, justification of high working stress, bonded or unbonded cables in prestressed concrete, and methods of prestressing the concrete. While Freyssinet was the first to recognize the need for high strength steel in prestressed concrete, Magnel was first to develop this idea in *Le Béton Précontraint* in writing.

2.2.1.3 The Need for High Strength Steel

Since designing prestressed slabs between two rock abutments is not practical, some other means of applying a force to a concrete member after it has cured must be developed. The most practical means of applying this force would turn out to be placing rods through metal ducts in the tension region of the slab. For the purpose of anchorage in Magnels' *Le Béton Précontraint*, consider a tie rod, which passes through the duct along the line of the previous 268,000 lb per foot (plf) (3,911 kN/m) force. As before, the shoring is constructed and the concrete is placed. For this example, the metal ducts are placed in the same position to which the previous force in Magnel's example was applied. The tie rods, or reinforcing bars with threaded ends, pass through the ducts to prevent them from bonding to the concrete. This allows a more uniform application of stress throughout the prestressing bars as well as throughout the concrete beam. Also, these bars must extend past the end of the beam to allow for the post-tensioning force to be applied. Once the concrete is cured sufficiently, nuts are tightened on the threaded ends of the bars to impose a post-tensioned force to the concrete beam assuming sufficient anchorage is provided. If the anchorage provided isn't able to resist the tensile force that is going to be transferred to the beam, the required compressive force in the beam will not be able to be achieved. Subsequently, the nuts are tightened until they have 16,000 lb. per square inch (62,320 kPa) of stress applied to them. Now, instead of the jack causing a post-tensioned force, the tie rods create this force.

The first problem with this process is the amount of prestress steel required to obtain 268,000 lb per foot of force. With the stress in the bars at 16,000 psi (62,320 kPa), 16.7 in^2 (108 cm^2), or 4.4 percent of the gross concrete area of prestress bars is required. In terms of mild reinforcing steel, ten # 12 bars are required along the beam. This is physically impossible, since these bars are inside the metal duct, which further increases the size. However the large quantity and high cost of this steel is not the major problem with using mild reinforcing steel for post-tensioning. Elongation of steel must be considered when placing a high tensile stress on low—strength, mild reinforcing steel. If a 66 foot (20m) long piece of steel is stressed to 16,000 psi (62,320 kPa), it will elongate approximately one-half inch (12.5 mm). When the concrete reaches its 28-day compressive strength, it is said to be cured enough to resist the post-tensioning force. In 1948, engineers used the 28 day strength, but now we tend to use 7 day strength because we have found through testing that the strength jump in the first 7 days is significant enough to resist prestress forces. But by 7 days, not all of the cement has hydrated, and the section has not completely cured, which means an additional one-quarter inch

(6.25mm) of shrinkage in the concrete will occur. Once the one-half inch (12.5mm) elongation of the steel takes place and the one-quarter inch (6.25mm) shrinkage of concrete occurs, the beam appears to be in its final position. This elongation and shrinkage causes one-half of the initial prestress remaining after the first few months post application. Secondly, creep should be considered. In the first few months after the concrete is cured, the beam will creep at least another one-quarter inch, losing all effect of the initial prestressing force. Notably, steel may only be pulled in tension until it elongates to a certain position. Once this point is reached, it will start to neck, or narrow in diameter, and only to a certain point until it fractures.

Creep in steel also needs to be considered since it will decrease the prestress force even more than concrete creep alone. To clarify, as long as steel is used than cannot be elongated more that one-half inch (12.5mm) in 66 ft (20m), establishing any prestress force in the concrete is impossible. Since shrinkage and creep cannot be controlled, because they are inherent qualities of concrete; one-half of the initial elongation will be lost automatically.

Given this phenomenon, very high strength steel bars that can safely resist up to 120,000 psi (827 mPa) must be used. Since all steels have approximately the same modulus of elasticity, or the same strain under the same stress, this would have been impossible. The only material that was available in 1948 was an 0.2 inch (5.08mm) cold drawn steel wire with a tensile strength of approximately 224,000 psi (1545 mPa) and requiring 160,000 psi (1,103 mPa) to give a permanent elongation of 0.2 percent. However, a safe stress for in design with cold drawn steel wire was 120,000 psi.

Now that the steel stress has been established as 160,000 psi (1,103 mPa) steel instead of the mild reinforcement, or 16,000 psi (120 mPa) steel, a calculation of the new required area of steel is possible. First, the loss due to creep and shrinkage will conservatively be taken as 15 percent. Next, the amount of steel to create a force of 268,000 psi (1,848 mPa), plus 15 percent for shrinkage and creep, is 2.57 in^2 (16.58 cm^2), which is approximately 88 – 0.2” (5.08mm) diameter wires per square foot of concrete. Because it would not be practical to place each wire in its own duct, these small diameter wires may be weaved together to form cables. Thus, eighty-eight wires can be made into two 44 strand cables each placed into its own duct. Each cable, duct included, does not exceed two inches (5.08cm) in diameter. Compared to the initial mild reinforcing design of ten 1 ½" (38.1mm) cables per foot width, two 2" ducts are minimal.

To recap the design of this bridge slab, the prestressed concrete is 2'8" (0.81m) deep while the mild reinforced concrete is 3'7" (1.1m) deep. The post-tensioned slab has 2.57 in^2 (16.54 cm^2) of high strength steel while the mild reinforced concrete slab has 11.4 in^2 (73.55 cm^2) of mild steel reinforcing.

2.2.1.4 Parabolic Curvature in Prestress Cables

Up to this point in this report, the design of post-tensioned concrete has only considered the stresses at midspan. In a simply supported member such as our bridge slab, the bending moment due to imposed loads is smaller at any section of the slab other than at midspan, and the compressive stress in the top fiber at any point other than midspan for dead load is less than 1,275 lb. per square inch (8,791 kPa). This also means that the stress due to dead load directly above the supports is equal to zero. The stress of 1,275 lb. per square inch at mid-span offsets to a certain degree the stress of 1,120 lb. per square inch (7,722 kPa) due to the applied prestressing force of 268,000 psi (1848 mPa) (See Figure 4). This condition suggests that with a straight orientation of the cable, the ends of the beam will develop high tensile stresses at the top fiber. To negate the effects of the prestress force on the ends of the beam, the cable may be oriented in a parabolic shape to match the moment diagram of the simply supported beam. For maximum potential of the prestressing/post-tensioning steel, the ducts can be oriented so that the greatest eccentricity is at midspan of the beam, while the eccentricity at the end of the beam can be left at zero to avoid additional tensile stresses at the ends of the beam. This can be thought of as negating the effects of the moment imposed on the beam with the following theory:

If the prestress force is acting in a parabolic manner with the greatest downward eccentricity at the midspan, the concrete stresses from the prestress force can be modeled as a mirror image of this parabolic shape. The concrete now has the maximum imposed compressive force at midspan with the force decreasing at the rate that the eccentricity in the cable decreases towards the ends of the beam. When the forces are imposed on the beam, the bending stresses act on the beam in the same shape as the moment diagram of the beam. If designed correctly, the prestress force will be sufficient at midspan that the bottom fibers are never allowed to go into tension, where cracking will occur, and the top fibers at the ends of the beam will also never go into tension. This design theory fulfills two main objectives of prestressed/post-tensioned concrete: It prevents cracking throughout the entire beam in both the top and bottom fibers, and it also resists all tensile forces throughout the entire section so no mild reinforcement is required for strength.

The prestress force applied to the beam also helps reduce diagonal tension at the slab ends. Raising the prestress cables to the middle of the slab at the ends introduces a shearing force opposite to the shear from imposed loads. Thus, the entire slab is subjected to the compressive force at the ends where the shearing force is greatest, thereby reducing diagonal tension. In Figure 2-8 Magnel depicts this parabolic shape of the cable.

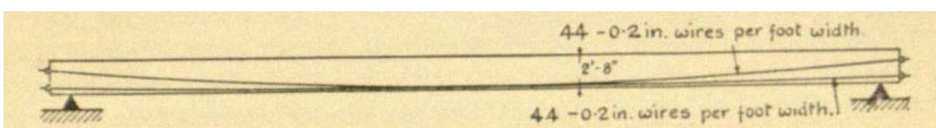

Figure 2-8 Parabolic shape in prestress cables (Magnel 1954)

2.2.1.5 Justification of High Working Stress

In 1948, very little research on prestress/post-tensioned concrete had been done; indeed, most acceptable design methods were empirical. However, today we are seeing a transition from empirical to strength design, and yet more common than this one is the transition from allowable stress design to ultimate strength design. Where concrete is allowed to crack, utilizing the steels' full capacity. This has allowed for much shallower, smaller sections because the full yield capacity of steel is allowed. In 1948, it was considered unacceptable to allow the stress in steel to rise above fifty percent of the yield stress. This was justified by the considerations that "a bar may have a local defect, that the bars are submitted to severe variations in stress resulting possibly in fatigue, and that the concrete, which cannot be strained to the same extent as the steel, cracks long before the working stress is reached; thus using bars of high tensile strength or high yield-stress is not justifiable if cracks are undesirable" (Magnel 1954).

In prestressed/post-tensioned concrete, these considerations are not valid. First, every strand is tested separately to a stress much higher than it will be subjected to, which is set by the designer. Later, only a small group of specimens would be tested from each design, but since this steel was so new, every specimen was tested. Secondly, so many wires are present that if one has a defect and breaks, the loss in tensioning is negligible. Since concrete only cracks in tension, unless crushing occurs because of too high a prestress force, prestressing/post-tensioning force puts the entire member in compression. This justifies imposing a stress in the steel of up to 80 percent of the yield stress, or up to 60 percent of the tensile strength, whichever gives the lower working stress.

Magnel does express reservation about the parabolic shape in his book. He states that whenever possible, a slight upward camber should be given to the beam so the cables can be straight. This eliminates the loss of prestress force due to friction between the cable and the duct through which it runs. Figure 2-9 displays Magnel's (1954) idea of linear cambering of the beam to avoid friction.

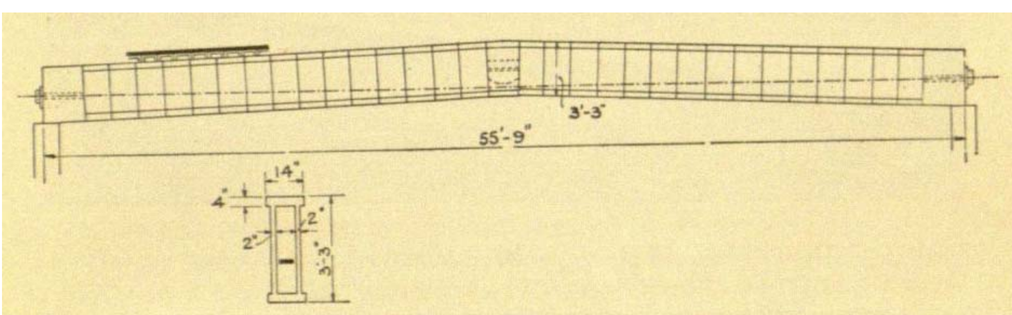

Figure 2-9 Cambered beam with straight cable (Magnel 9)

2.2.1.6 Bonded Cables in Prestressed Concrete

In the previous example, the prestressed force could be obtained by another method. A steel plate containing a duct for the prestressed wires to fit through is set at each end of the beam. These steel plates must be able to resist the induced compressive prestress force in the cables. The wires are similar to the wires that passed through the ducts in the previous example, but they are now 0.08 inches in diameter and of an even higher yield stress (280,000 psi) (1931 mPa). These cables are secured to one end of the beam at the plate and are stressed at the other end by a specialized jacking device, which bears on the other steel mold. The wires are held in place by an anchorage device while the concrete is placed into the formwork and is cured to a sufficient compressive strength. After the concrete is cured, the anchorage devices are freed, and the wires attempt to regain their initial position. Accordingly the tensile forces in the steel wires are transferred to the concrete by the bond between the steel wire and the concrete.

The design of the same sized beam with the same number of cables can be altered significantly solely by changing the induced stress on different cables throughout the section. With a line of cables from the centroid of the section down to two inches from the bottom of the beam, the bottom cables would be those stressed the most, and the stress would decrease in a parabolic manner until the cable at the centroid would have zero stress in it. This would provide the same results, provided sufficient bond and sufficient calculation of loss of prestress through bond, as the parabolic profile discussed earlier.

2.2.1.7 Methods of Prestressing Concrete

In 1954 as prestressed/post-tensioned concrete was evolving, the most economical and user—friendly jacking and anchorage systems were being designed. Globally, different systems appeared, and each designer modified his or her system to fit his design style. In 1928, Eugene Freyssinet was the first to come up with a system of prestressing followed by Gustave Magnel's 'Belgium Sandwich Cable System' in the 1930's. Many other systems were being developed in the 1950's including a couple from the United States, discussed in the following sections, based on the two common ways to stress steel in concrete: 1) Prior to concrete being placed or prestressed, and 2) After concrete is cured or post-tensioning.   Notably, both of these methods are considered prestressed since the steel is stressed prior to any superimposed loading. For this report, I will use the term post-tensioning for stressing the steel after concrete is cured and prestressed for stressing of the steel prior to the concrete placing.

2.2.1.7.1 Post-Tensioned: Cables Tensioned after the Concrete has Hardened

Many of the projects at this time were 'one-of-a-kind,' so plant production of typical beams was not very practical. Also, when the cables are stretched after the concrete is hardened, many disadvantages can quickly occur, such as: loss of prestress due to friction, concrete bond, anchorage, and creep.

2.2.1.7.1.1 "M. Freyssinet's Method" of Post-Tensioning

In 1928, Freyssinet utilized 0.2 inch or 0.276 inch diameter, high strength wires (See the patent in Appendix). Usually ten to eighteen wires formed a prestress cable, and these cables were allowed to be stressed to about 120,000 psi (827 mPa), which results in prestressed force of 25 tons and 50 tons (222 kN and 444 kN) from 0.2 inch and 0.276 inch (5mm or 7mm) cables, respectively. Freyssinet's original method placed these wires indiscriminately in the cables. He then placed exactly equal prestress on each of the cables by hydraulic jacking. Figure 2-10 shows Freyssinet's method. In 9(a), the helical spring can be seen with the wires wrapped around it. Before the cable reaches the helical spring, it passes through an anchorage device consisting of an extractable rubber core that is formed into the concrete with a central hole through which the cable passes. For unbonded post-tensioning, the cable wrapped in bituminous paper and laid in the formwork or sheathed metal and laid in the formwork. Currently, a system wraps the wires around a helical spring that is outside of the beam dimensions.

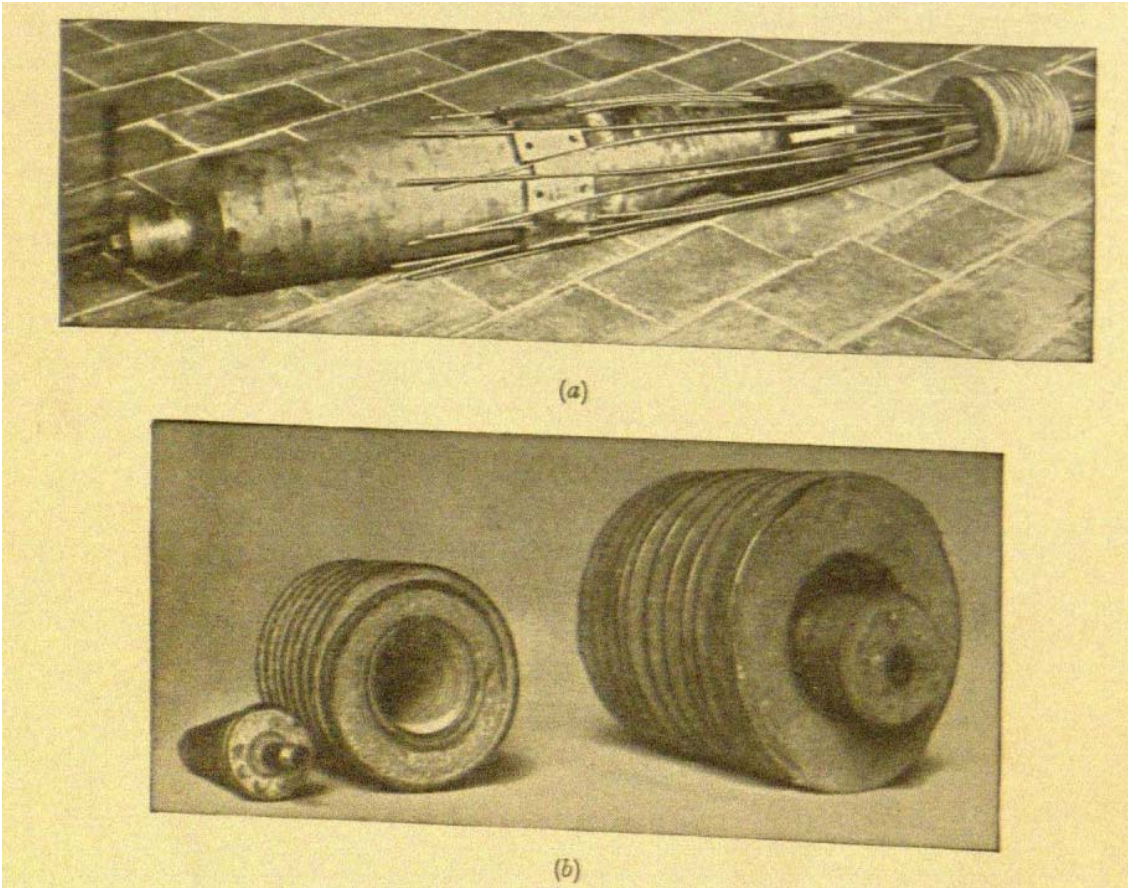

Figure 2-10 M. Freyssinet's Method (Magnel 1954)

These high strength wires are held at the end of the beam by cylindrical blocks made of a mortar that is reinforced with a steel hoop. The blocks are included in the concrete formwork and are actually poured into the beam after it is cured. The wires pass through these cylindrical blocks in a central hole, which is plugged by a "rich mortar plug reinforced with fine wire (Magnel 1954)." After the concrete is cured, the wires are jacked at one end and temporarily fixed to the jack by steel wedges. When the jack pulls the wires to the required tensile stress, the plug is pressed to its final position by a ram that extends from the jack. These plugs hold the wires in place and retain the tensile force in the wires, which is transferred through the system into the concrete.

- Advantages of this system include the following: the securing of the wires is not expensive; the stretching force is obtained fairly quickly; the mortar blocks may be left in the concrete; and, the mortar blocks do not protrude beyond the ends of the concrete.

- Disadvantages of Freyssinet's method are as follows: The stretching of all the wires of a cable at once may not produce the same stress in each of the cables; the shape and quality of the end blocks may not be uniform; the maximum stretching force is 25 tons to 50 tons; the force required will be much more, even for a small bridge; the jacks are heavy and expensive compared to those needed when two wires are stretched at a time.

2.2.1.7.1.2 "The Belgian Sandwich Cable System" of Post-Tensioning

During World War II, it was impossible for Gustave Magnel to obtain Freyssinet's devices. Freyssinet was working in France at this time, and the Germans were occupying Belgium where Magnel was sequestered. With the occupation of Belgium by the Axis powers, Magnel was certainly not allowed to have any correspondence with a French designer. This forced Magnel to design his own method of prestressing, seen in Figure 2-11, which he named the 'Belgian System' (Magnel 1954). This method's patent is in the Appendix. The principles that Magnel based his design on are innovative very important. (1954):

- The wires must not be placed randomly in a cable, but must be in a definite order.
- Between all wires in a cable, spaces of about 3/16 inch should be left to allow easy injection of cement grout to protect the wires from corrosion
- Only two wires should be stretched at a time, to that practically uniform stress results.
- The anchorage must be strong enough to permit an occasional defective wire to break when the stretching force in applied without damage to the locking device or release of the other wires.

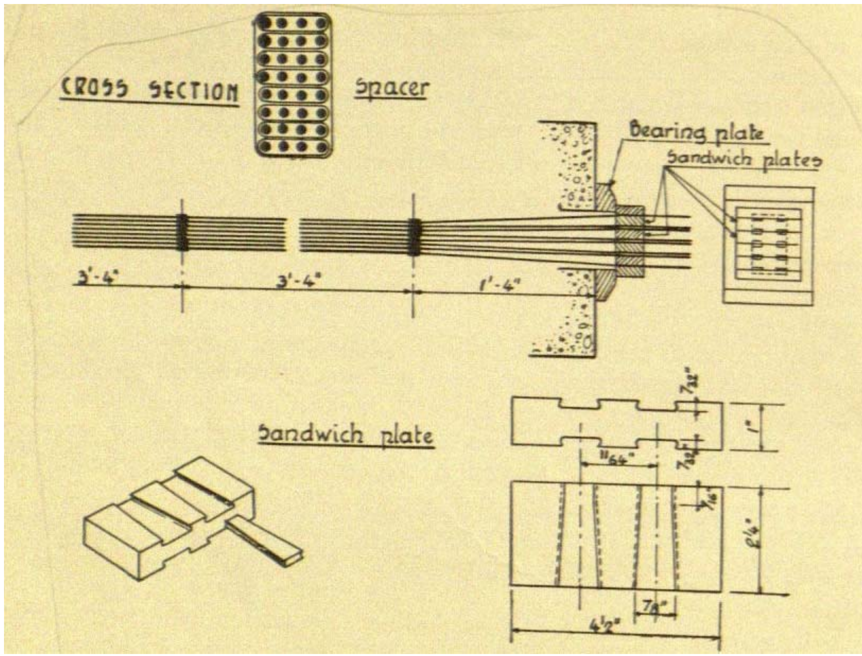

Figure 2-11 The Belgian Sandwich Cable (Magnel 1954)

This is a unique system. Figure 2-11 depicts how the sandwich cable works. Throughout the cross section the wires are set horizontally in groups of four. An average cable comprising of 32 wires would be oriented in eight layers of four cables. To keep the cables in the correct positions throughout the beam section, spacers would be provided both vertically and horizontally. These spacers are shown in the bottom left of Figure 2-10. Before the concrete is placed, the cables are either put in a sheet metal duct, or cylindrical holes are cast in the concrete so the cables can run through the beam after the concrete has hardened. “Each locking plate, called a “sandwich” plate, has four wedged-shaped grooves in each of which two wires are secured with a steel wedge” (1954). Then the cable is stressed, two wires at a time, by a 10-ton (89 kN) jack.

One of the advantages of this system is that the cables comprised a large number of smaller wires. “Cables comprising 64 – 0.2 (5mm) inch wires capable of applying a compressive force of 107 tons (952 mN) have already been made (in 1954), and in actual structures, cables of 64 – 0.276 inch (7mm) wires, capable of applying a compressive force of 214 tons (1905mN), have been made. (Magnel 19545)”

- The disadvantages of the Belgian System follow: It is more expensive than Freyssinet's system; it requires to stress the cables; the sandwich plates extend past the concrete edges; it is easier to drape cables at an angle from the center of the beam to the ends with Freyssinet's system; Magnel believed that cables should not be angled towards the center of the beam because stretching a cable with an angled profile results in frictional resistance, which in turn reduces the required elongation under required force.

One specific feature of Magnel's Belgium System is the type of jack, which no longer required the ram to drive the wedges into the anchorage, as did Freyssinet's system.

2.2.1.7.1.3 "The Franki Method" of Post-Tensioning (Belgium)

Mr. Franki invented the Franki Method, which is a combination of M. Freyssinet's Method and Magnel's Sandwich-Cable Method. Here a steel tube encompasses 12 wires, as in Freyssinet's method, of either 0.2 inch (5mm) or 0.276 inch (7mm) diameters, which are held apart by steel spacers, such as the sandwich cables. The anchorage consists of steel plates with twelve conical wedges each holding one of the twelve wires. Just as in the sandwich cable system, the tensioning is achieved by stretching two wires at a time to the required tensile stress.

2.2.1.7.1.4"Electrical Prestressing" Post-Tensioning

Mr. R. M. Carlson and Mr. Billner had a different, very interesting, idea for prestressing. They used steel bars, much like mild reinforcing steel, which can be safely stressed to 28,000 psi (193 mPa). The steel bars were threaded at the ends and coated with a solid layer of sulphur by dipping the steel in a bath of molten sulphur. When the bar returns to normal room temperature, the sulphur solidifies and coats the steel. Then the bars are placed in the concrete just as mild reinforcing steel is, but the threaded ends of the bars extend beyond the end of the beam. After the concrete reaches required strength, the bars are connected to an electrical current. According to Magnel (1954), the bars are heated for 2 minutes with 5 volts for every three feet of bar. The other criterion for heating was the bars are subjected to 400 amperes of current for every 0.15 square inches (0.96 cm^2) of cross-section area of steel. The resistance of the electric current in the steel creates heat as a byproduct of energy; enough to melt the sulphur, breaking the link between the steel and the concrete. This amount of heat also elongates the steel enough to produce a tensile stress in the steel. Once elongation occurs, nuts are tightened on the threads of the bars, which extend past the concrete edge. The tightened nuts provide resistance against the concrete to keep the bars in tension. The electricity can then stop; once again, the sulphur hardens and produces a bond between the concrete and the steel. Once the bond is established, the nuts can be loosened transferring the tensile stress from the steel to the concrete through the bond between the concrete and steel.

Many disadvantages are clear: first, a large quantity of steel is wasted; then the ends of the bars must extend past the end of the concrete element; also, the entire cross--sectional area of the steel may not be used because the threads at the end have less area than the prestress steel. The only solution to this would be to construct the ends of the bars where the threads are, out of thicker steel so the entire cross section of the embedded steel could be used. Another disadvantage is that the designer has no way to determine if the prestress force is uniform throughout the whole cross section. Also, the engineer did not know if the chemical reaction of the sulphur was damaging to the concrete, the steel, or the bond between steel and concrete. Finally, once the sulphur is liquefied, the possibility of moisture in the concrete exists.

2.2.1.7.1.5 "K. P. Billner's Method" (USA)

K. P. Billner proposed a different method of prestressing requiring concrete to be cast in two different molds. The molds were separated at midspan offering the prestress wires as the only connection between the two beams. The wires were coated with asphalt except at the ends, passed through the end of the beam and were fixed by loops, which concreted in the end blocks. Once the concrete had cured to a required strength, the beams were pulled apart by two jacks acting against steel plates that were cast in the inner parts of the half-beams. The asphalt created a barrier between the concrete and the steel, so no bond existed that would allow the steel to elongate. Once the jacks stressed the steel wires to desired elongation, a rich ‘quick-set’ mortar containing calcium chloride was placed between the beams and allowed to set. The calcium chloride helped speed up the chemical process of within a few hours, curing after which the jacks could be removed once the desired prestress force was achieved.

2.2.1.7.1.7 “Freyssinet’s Flat Jacks”

Freyssinet’s flat jacks are composed of two parallel flat steel plates with a small space between them. The ends of these steel plates are joined by another steel plate shaped like a torus, which looks like a barbell in that it is almost flat at the center and has round hollow balls at the ends. A pressure nozzle is fixed to one of the end balls, allowing a pressurized liquid to be pumped into the hollow space, and thereby causing the inside of the steel in the middle portion to expand. Figure 2-11 shows this shape and where the pressure nozzle is inserted. This system works very well in regions where bridges or other structures span across points with rock abutments on either side of the member. First, the concrete is cast while the steel is in its depressurized state. Once the concrete has cured, the pressurized liquid is introduced to the steel shape at the ends of the member and after expansion, compression is introduced in the concrete member.

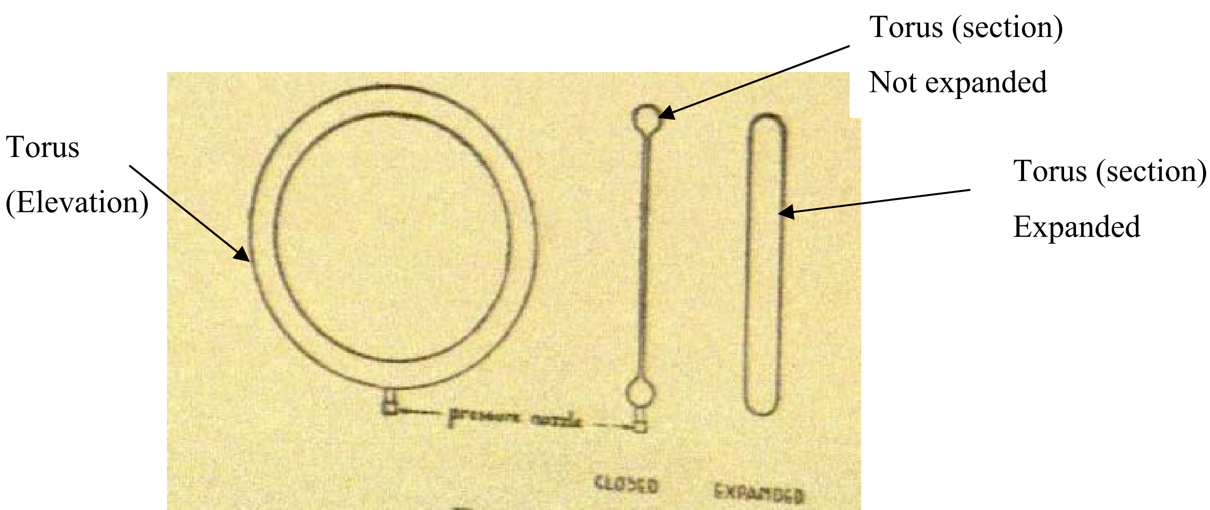

Figure 2-12 Freyssinet's Flat Jacks (Magnel 1954)

2.2.1.7.1.8 “Lee-McCall System” (Great Britain)

The Lee-McCall system is composed of high strength steel bars instead of high strength wires whose ends are threaded similarly to the bars in the electrical prestressing method. The bars are placed in the member just as mild reinforcing steel would be placed, and once the concrete has cured, nuts are placed on the threads. Steel plates are included on each end of the members for the nuts to bear against when tightened to prevent the steel nuts from crushing the concrete locally instead of tensioning the steel bars. The Lee-McCall system was considered “acceptable” (1954) by Magnel as long as the ends of the bars did not produce sharp angles of high stress concentrations at the bent sections. It was important for Magnel to consider a method other than his own acceptable in a book published worldwide since designers might be looking for other methods that they could use.

2.2.1.7.1.9 “Dr. Leonhardt’s Method”

Dr. Leonhardt’s prestressing method was used in many very “important works” in Germany according to Gustave Magnel (1954). Leonhardt was able to stress high strength steel wires to develop very high tensioning forces in the range of several thousand tons (>8900 mN). The wires were actually doubled over so at one end of the beam the cable produced a loop and at the other end of the beam were the two end wires. The looped end of the wire curved around a cylindrical surface that was separate from the structural member. As shown in Figure 2-13 (a), the two free ends looped around another end block, cast with the beam, to provide anchorage. Next, jacks were placed between the end block which was separated from the beam, and the looped center section of wire to further separate that block from the beam. The jack cylinders were cylindrical openings formed in the end block with steel plates. The pistons of the jacks were also cast in the concrete in steel forms. The jacks were kept water-tight by rubber sleeves. Once the jack separates the end block from the beam enough to achieve required elongation in the steel wires, the void between the end block and the beam was concreted, with minimal shrinkage concrete, to keep the tensile force in the steel constant. One disadvantage of this method is the pistons and the cylinders of the jacks are lost in the concrete and cannot be reused.

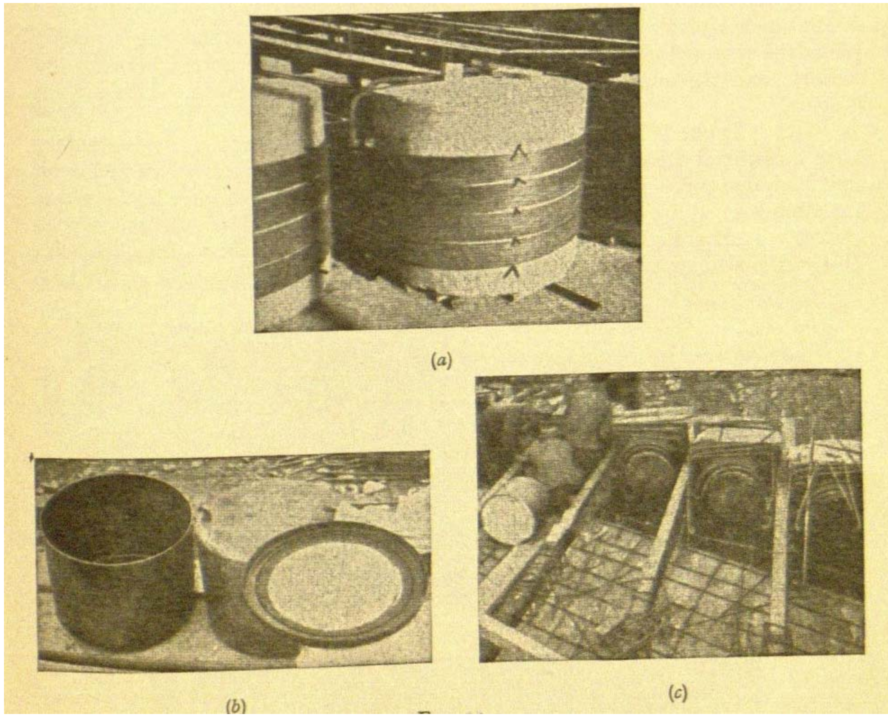

Figure 2-13 Dr. Leonhardt's Method (Magnel 1954)

2.2.1.7.1.10 “Professor Ros’s Method” (Switzerland)

Professor Ros in Switzerland created a method to prestress or post-tension using 0.2 or 0.276 (5 or 7mm) inch high strength steel wires. First, he enlarged the ends of the wires to the shape of a nail head and threaded cables through equally spaced holes in steel plates, and then enlarged the ends of the wires. One end of the beam was comprised of this cable and steel plate combination anchored to the beam end; the other end was fixed to a large jack that pulled the plate and wires away from the beam. This provided the required elongation to achieve a large tensile force in the steel wires. Once elongation was achieved, steel blocks were inserted between the beam end and the jacked plate, and then the jacks were removed. According to Magnel, “This system is sound.” (1954)

2.2.1.7.1.11 "The 'Leoba' Method" (Germany)

Mr. Leoba of Germany created a method for prestressing named the 'Leoba' method that used twelve 0.2 inch (5mm) high strength steel wires encased in a metal sheath. As Figure 2-14(a) shows, one end of the wires was jacked hydraulically to provide the tensile force. The fixed end, in Figure 2-14(b), was constructed by bending the ends of the wires. To reinforce the anchorage zone to prevent very high local stresses, a steel helix surrounded and tied to the bent wires. At the tensioning end shown in Figure 2-14(c), the wires looped around a steel cross-head. The center of the steel cross-head was threaded to allow a threaded bar to be screwed through it. The tensioning end was also surrounded by a steel helix to provide extra anchorage. Meanwhile, the tensioning end of the cable was held by a rubber sleeve to the left of the cross-head in Figure 2-14 (c), in the enlarged end of the sleeve. Next, a bolt was fixed to the cross-head that passed through the rubber sleeve, and the formwork and was held by nuts. After the concrete had hardened, the bolt, nuts, and sleeve were removed, and a threaded rod was screwed into the cross-head. The threaded rod passed through a washer and then a steel plate placed at the end of the beam. When the tensioning was taking place, a nut was tightened on the threaded rod to provide anchorage when the jacking device was removed. The end of the jack rested on the steel plate at the end of the beam, and the tensioning ram was attached to the threaded rod to provide required elongation in the high strength wires.

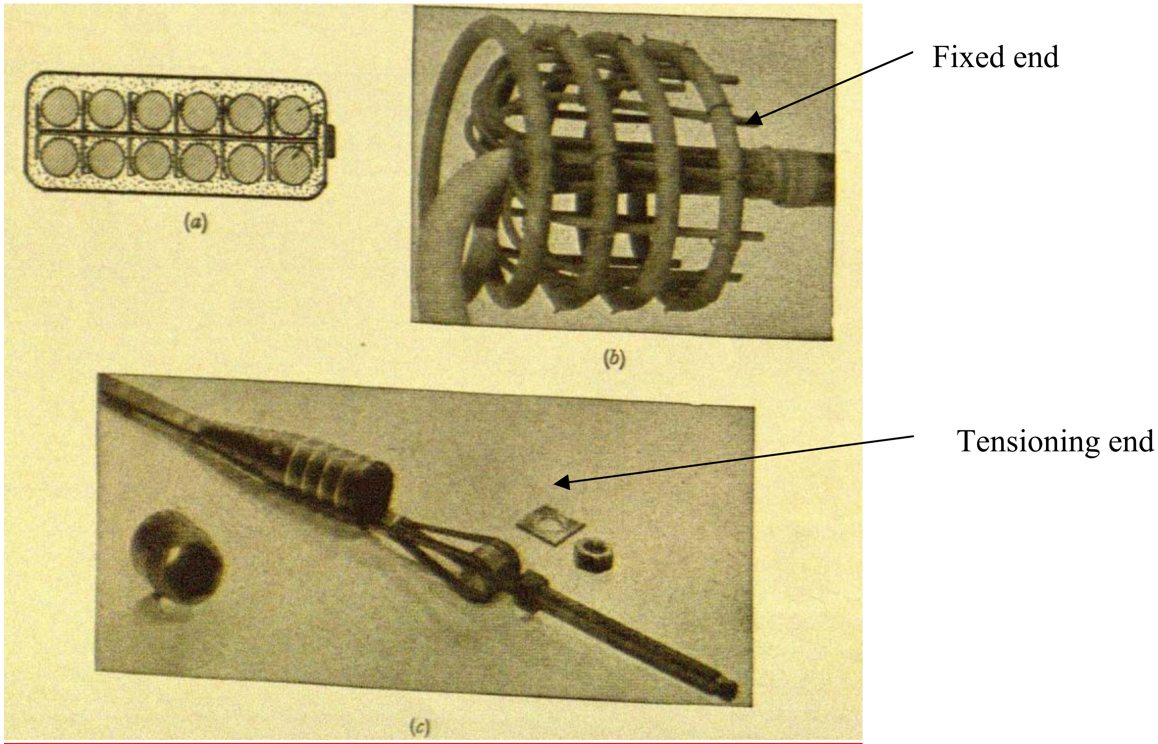

Figure 2-14 The "Leoba" Method (Magnel 1954)

2.2.1.7.1.12 “The Dywidag System“Stahl 90”” (Germany)

Urlich Finsterwalder created the Dywidag System “Stahl 90” in Germany, which utilized high strength steel bars instead of wires. “Stahl 90” is a tempered, naturally hard steel with yield point of 93,000 psi. The ultimate strength is 128,000 psi (883 mPa) and the creep limit is 78,300 psi. Average elongation is 14 percent. (Finsterwalder 1952)” “In the Dywidag process, “Stahl 90” is used because of the stresses on the transition from serviceable load to 1.75 times serviceable load, or 64,000 psi (441 mPa) to 93,000 psi (641 mPa), meaning an increase of 29,000 psi (200 mPa). Considering a rod elongation of 0.1 percent, no more unfavorable cracking would occur than with normal (reinforced) concrete.

Finsterwalder (1952) noted that the losses in prestress due to shrinkage and creep are somewhat more than for high strength wires, but this is inconsequential because this method uses limited prestressing.

Notably, the threads of the bars are cold rolled to make the end thread section of the bar the same strength as the middle of the bars. Next, the ends pass through a steel end plate at the end of the beam. Whereupon, the bars are cast into the beam in 1 ¼" ducts. While the bars are being jacked, the nuts are tightened on the bar to provide anchorage. Finally, once the bars are pulled to required elongation, the ducts are filled with grout.

2.2.1.7.2 Prestressed: Cables Stretched before Concrete is Placed

Designers in Europe quickly saw an advantage to mass producing certain 'standard' shapes and lengths of prestressed concrete beams. However, one disadvantage was that the designer was more restricted by the shape and size of the members chosen. In contrast, one important advantage is that prestressed manufacturers are able to produce many, often many hundred, beams per day, and formwork is often temporarily reusable.

2.2.1.7.2.1 "Hoyer's System" of Prestressing

Mr. Hoyer in Belgium began producing precast prestressed concrete elements using the Hoyer System of two buttresses fixed 300 foot (91.5m) from each other with high strength steel wires stretched between them and formwork placed on each side of the wires. For instance, if the designer desired fourteen 20 foot (6m) long beams, the formwork could be placed at 20 foot on center with a few inches between each of the forms. Once the formwork is in position, the concrete is placed and consolidated, and after sufficient cure, the wires can all be cut apart and the beams separated. As Magnel explains "…using twenty similar lines of manufacture, 280 beams can be in production at the same time. If the time required for the hardening of the concrete is four days, 70 beams can be made every day (Magnel 1954)" By rotating the beams in increments of four days, the plant can be building formwork, stressing cables, pouring concrete, and stripping forms for different beams all in one day. This keeps the workflow constant and at this time would have been considered a more Americanized method compared to the others because it resembles an assembly line. Figure 2-15 shows a factory in Belgium where beams like this were made.

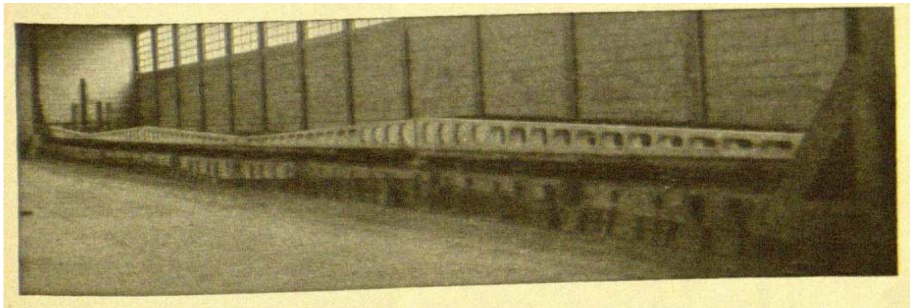

Figure 2-15 Precast beams in a factory in Belgium (Magnel 1954)

2.2.1.7.2.2 “Schorer’s System” of Prestressing

Gustave Magnel found Schorer’s System very interesting because it did not use any permanent fixing devices - anchorage devices were a main reason in the high cost of prestressed concrete. This system is shown in Figure 2-16. Unfortunately, he was never able, up until the date of *Prestressed Concrete*, to test this system,

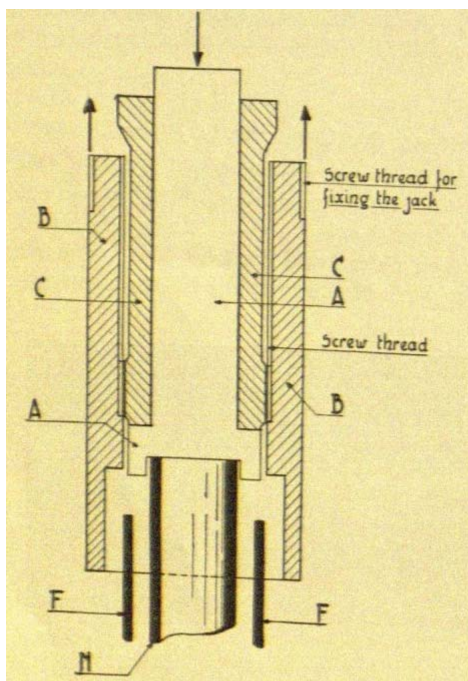

Figure 2-16 Schorer's System (Magnel 1954)

Schorer, much like Hoyer, induced the compressive stress in concrete through the bond between concrete and high strength wires. He utilized 0.08” (2mm) and 0.11” (2.8mm) wires with yield stresses from 190,000 (1310 mPa) to 220,000 psi (1,517 mPa). These wires produced comfortable working stresses of 120,000 (827 mPa) to 145,000 psi (1,000 mPa). One of the disadvantages of Hoyer's method is it requires very stable end buttresses, which Schorer created by wrapping wires helically around a central, high tensile strength, steel tube capable of resisting a compressive stress of 100,000 psi (670 mPa). This tube's main purpose is to resist the pull of the jack as the wires are being tensioned. Magnel (1954) in *Prestressed Concrete* shows the resisting capacity of this tube in the following example, "…for a cable which has to produce a force of 10 tons (89 kN), the tube would be about 1 inch in diameter with a wall thickness of about 5/64 inch (2mm), and 32 wires of 0.08 in (2mm) diameter would be required." The tubes resist buckling by the resistance to deviation of the tensioned wires. After the wires are stressed they are anchored, and the tube is pulled out from the middle of the wires.

The process of stressing the wires and creating the prestressed concrete beam is as follows. The tube is prevented from bonding to the concrete by either a paper or sheet metal wrap on the outside surface to allow movement after which the wires are wrapped around the tube spirally with a very wide pitch. Half of the wires wrap in one direction while the other half wrap in the other direction. The wires are held away from the tube by small 3/16" thick disks on the outer face of the tube and are temporarily secured to the device by steel wedges. Figure 2-16 shows this device, which essentially consisted of two parts that are able to slide, one along the other. The core, part A, rests on the high strength steel tube, and the wires are attached to part B or the crown by the steel wedge aforementioned. The tensioning jack rests on the core and pulls the crown pulls away from the core to achieve desired elongation, and the two parts are fixed to one another by means of nut C. After nut C is tightened and fixes the two pieces together, the jack may be taken away. The compression tubes are designed to resist forces of between 2 and 20 tons (17.8 kN – 178 kN). In addition to resisting compressive forces from the wires in tension, the tubes also act as reinforcement. Meanwhile, the compressive force is transferred from the wires to the concrete beam by bond of the wires to the concrete. In fact, the bond in Schorer's method is much better than in Hoyer's due to the spiral wrapping of the wires around the central tube. Once the wires are bonded to the concrete, the tube may be reused on the next beam. The hole left by the tube may then be easily filled with high strength low shrink grout to resist corrosion of the steel wires.

Magnel (1954) says that the inventor, Schorer, tested this beam and it behaved very well. However, Magnel was concerned about the small amount of mortar, approximately 3/16 inch (4.8mm), which surrounded the wires believing that its strength may not be sufficient to resist the large loads applied to prestressed beams.

2.2.2 The Walnut Lane Bridge the First Major Prestressed Concrete Structure in the United States

Magnel's drive for simplicity in formulas and explanations as well as his accumulated experience in reinforced concrete and prestressed concrete helped him gain credibility in the field. In 1948, the opportunity to use his knowledge in a practical application in the U.S.A. came about when American engineers in Philadelphia turned to Gustave Magnel to design the first major public structure out of prestressed concrete.

The design and construction of the Walnut Lane Bridge in Philadelphia, Figure 2-17, would prove to be the greatest structural engineering feat of prestressed concrete in history. In a speech at the First United States Conference of Prestressed Concrete, Samuel S. Baxter stated that had the original arch design for the new Walnut Lane Bridge been bid below the engineers' estimate (Billington 2004):

"It is also quite possible that this First Conference on Prestressed Concrete might not now be in session…"

Figure 2-17 Walnut Lane Bridge (Janberg 2009)

Without a doubt, prestressed concrete would have made its way eventually into the United States, and most likely there would have been some conference for Prestressed Concrete. However, the Walnut Lane Bridge would still characterize the potential of prestressed concrete because of its large scale spans, 160 foot main spans (49m), construction economy and most importantly, its acceptance by city engineers as well as by the powerful Art Jury (Billington 2004). These were the two groups of people normally associated with very conservative mind-sets, so their acceptance was vital in forwarding the development of prestressed.

The first design for the bridge consisted of a stone faced arch with primitive reinforced concrete as the structural element, which obtained a low bid almost $150,000 over the engineers' estimated low bid. By law, if a low bid exceeded the engineers' estimate, the low bid would be rejected so the search for another solution began. The first solution was to remove the stone facing which would prove to lower the low bid by $500,000, however the Art Jury firmly rejected this solution. The second solution came almost by accident.

At that time, The Bureau of Engineering, Surveys and Zoning was constructing large circular sludge tanks using the prestressing technique of winding wires around a thin core to achieve required strength of the concrete tanks. It took only one remark by Mr. E.R. Schofield, Chief of the Design Division for the Bureau for the city to begin exploring the possibility of a prestressed concrete design for their bridge. Consequentialy, Mr. Schofield was solicited as well as a new name in the engineering field in the United States, Professor Gustave Magnel in Belgium. After reviewing several designs and design philosophies, the city decided to choose the Preload Corporation out of New York, a firm specializing in 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 (Marianos 2005) where he would utilize his ideas for prestressed concrete girder design.

One small setback was convincing the Art Jury of the design's practicality given the Art Jury's conservative values and beliefs. Billington (2004) explains the Art Jury's response to Magnel's design as "one of the most historically significant events in the relationship between structure and aesthetics." The Jury's response is summed up with the following: "The Art Jury, however, on seeing the preliminary sketches for the new bridge agreed that the comparatively slim lines of the new bridge would not require stone facing." Thus, Philadelphia's most elegant natural park, Fairmont Park was able to house a major historical structure because, "its appearance was pleasing enough to permit it to be economical (Billington 2004)."

One of the features of bridge design agreed upon was the full-scale destruction of one of the 160 foot long girders. . Even though design documentation proved that these girders were sufficient to carry the loads, the test provided visual proof to hundreds of engineers of the overload capacity of the bridge that was built with this new design philosophy.

Figure 2-18 shows the cross-section of the bridge, and Figure 2-19 shows the testing of one of the 160’ bridge girders.

Magnel was astonished by the number of difficulties that arose when he tried to get American companies to manufacture his special fittings. He was used to labor and manufacturing processes proceeding smoothly and being cheap. In contrast, American manufacturers were reluctant to spend valuable work hours on extra products that were not ‘needed’ to complete the project. They only wanted to manufacture fittings that they knew would make them money.

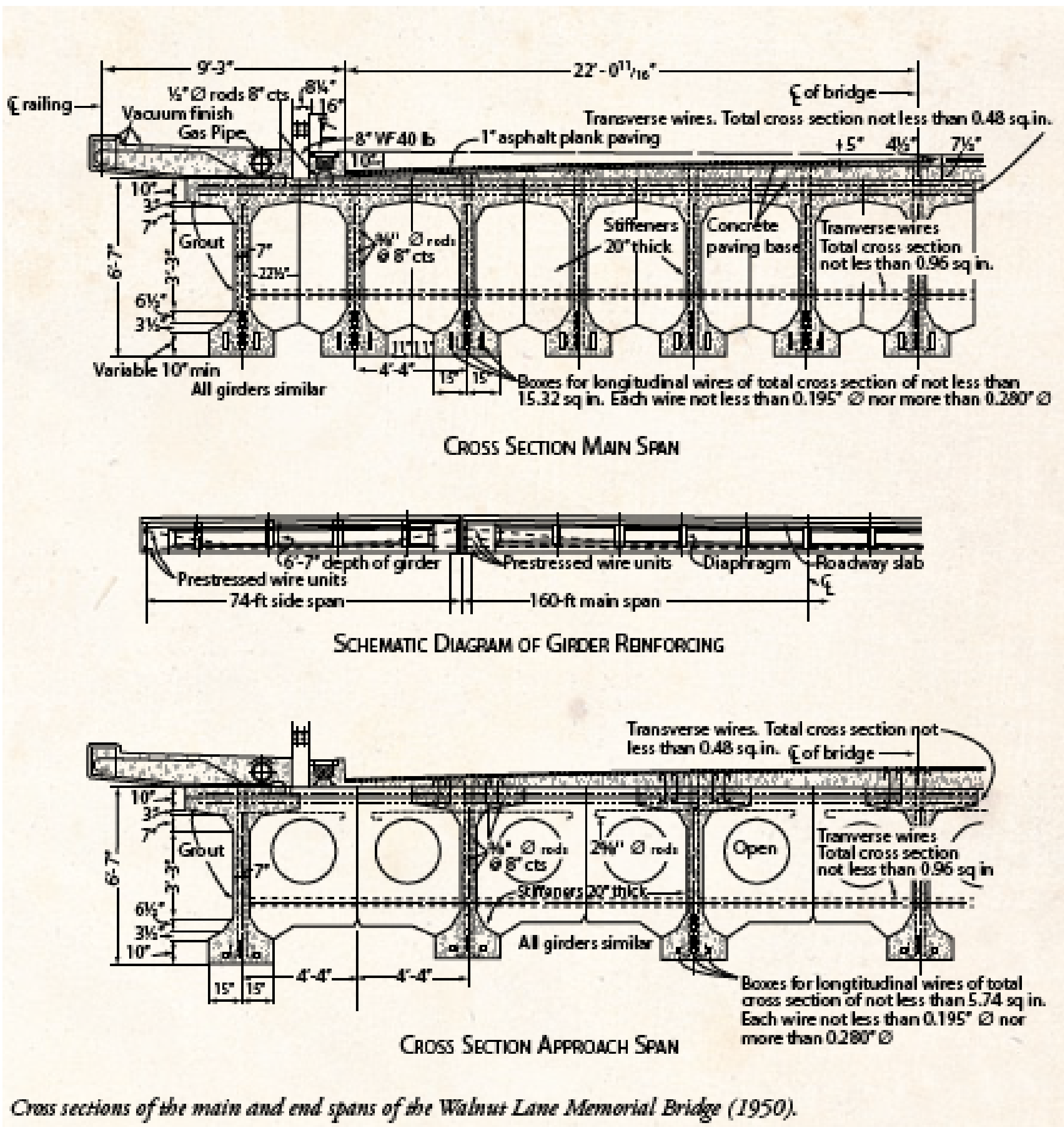

Figure 2-18 Cross Sections of the Walnut Lane Memorial Bridge (Nasser 2008)

After completion of the Walnut Lane Bridge in 1951, American engineers realized the need to develop their own method to produce prestressed concrete. Charles C. Zollman said, “It became apparent that, regardless of its merit, the concept (of prestressed concrete) could not be used extensively on this continent (North America) in its European form, as such, it was simply not competitive with other available materials (1980).” He says also that prestressed concrete was simply not competitive in the construction market with other building materials, namely steel.

“The reason was that European and American construction philosophies were diametrically opposed.” (Zollman 1980) In Europe, each engineering and construction project were designed and built as a “custom-made venture.” In this construction philosophy, the outcome of the project’s design was the only important objective. Ultimately, the amount of labor, time and money needed to construct a project was thought of as secondary to the design.

Alternatively, the Americans prided themselves on their assembly-line philosophy which yielded dramatic economic results in the 1950’s. As stated earlier, this was the golden age of capitalism in America and the economy was flourishing. The invention of the assembly line, or mass production, by Henry Ford in 1913 greatly changed Americans views on how large scale production should be achieved and it turn, further helped the American economy flourish in this economic era.

Gustave Magnel was well aware of this difference in design and construction philosophy. In fact he made a bold statement at the Canadian (Toronto) Conference on Prestressed Concrete:

“...In the United States, industry is developed in a wonderful way…This is due in part to an internal market of 160 million people…This has made possible the enormous development of mass production and the introduction of highly specialized labor saving machinery…Unfortunately, in bridge building, one cannot apply the idea of mass production…" (Zollman 1980).

Gustave Magnel was doubtless brilliant, but for once he was incorrect. Magnel's comment on the importance of mass production was right on the money, but what he didn't envision was the ingenuity, power, and capabilities of American engineers. He stated that mass production of prestressed structural elements capable of carrying high loads over long spans, such as in bridge construction, was impossible. In fact, while he was making this statement in Canada, Americans were working already on mass producing new anchorage for a new "Americanized" prestressing system. To understand why Magnel made these remarks, one has to understand that he was from the small country of Belgium, which is about the size of our smallest state, Rhode Island. In small European countries such as this, large concepts were envisioned. Thus, he saw no need to mass produce prestressing elements if every project was a customized. Accordingly, Magnel held on to the idea that pretensioning meant bond by the smooth, 2 mm max diameter then in use in Europe (Zollman 1980, p. 127).

Figure 2-19 shows a full scale testing to destruction of one of the bridge girders of the Walnut Lane Bridge. Figure 2-20 depicts an elevation of the bridge girder that includes the parabolic and straight prestress cables. It also shows a cross section at mid-span of the bridge, which identifies where the duct sheaths are for the prestress cables at mid-span. Figure 2-21 shows a photograph of the construction of one of the girders. Much of the mild reinforcing steel is shown, and the rubber sheaths for the prestress cable are shown as well. Figure 2-22 shows a drawing of the bridge elevation in Magnel's *Prestressed Concrete.*

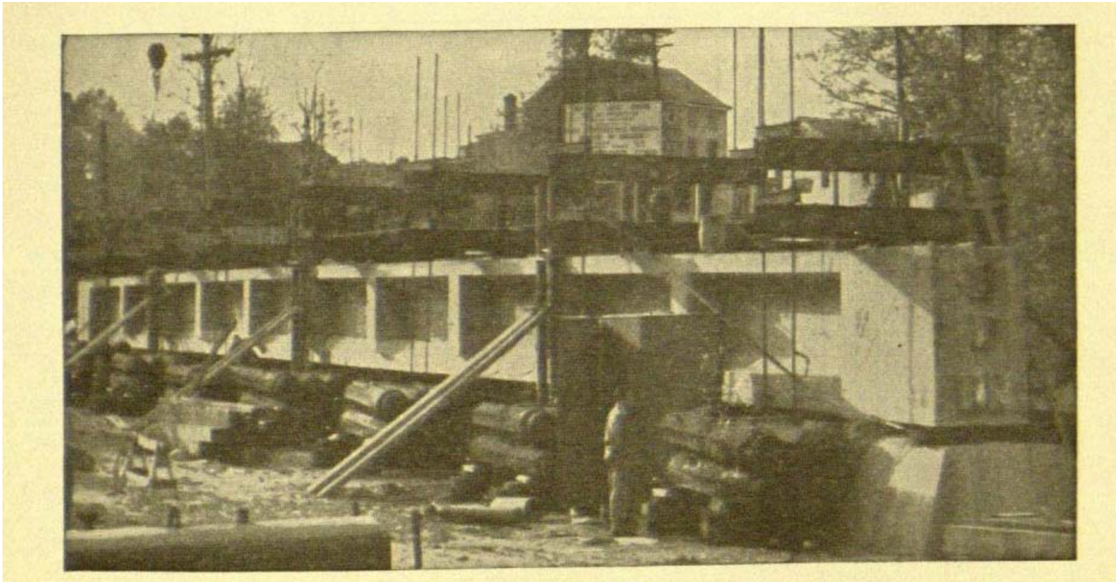

Figure 2-19 Walnut Lane Bridge Beam Test (155ft Span) (Mangel 1954)

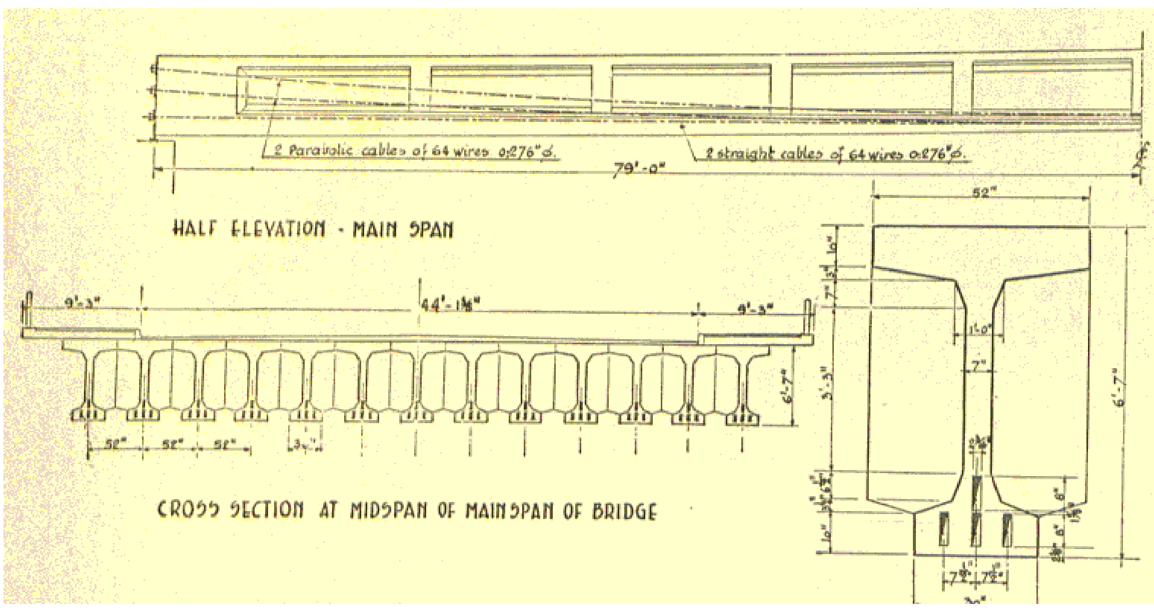

Figure 2-20 Walnut Lane Bridge Details (Magnel 1954)

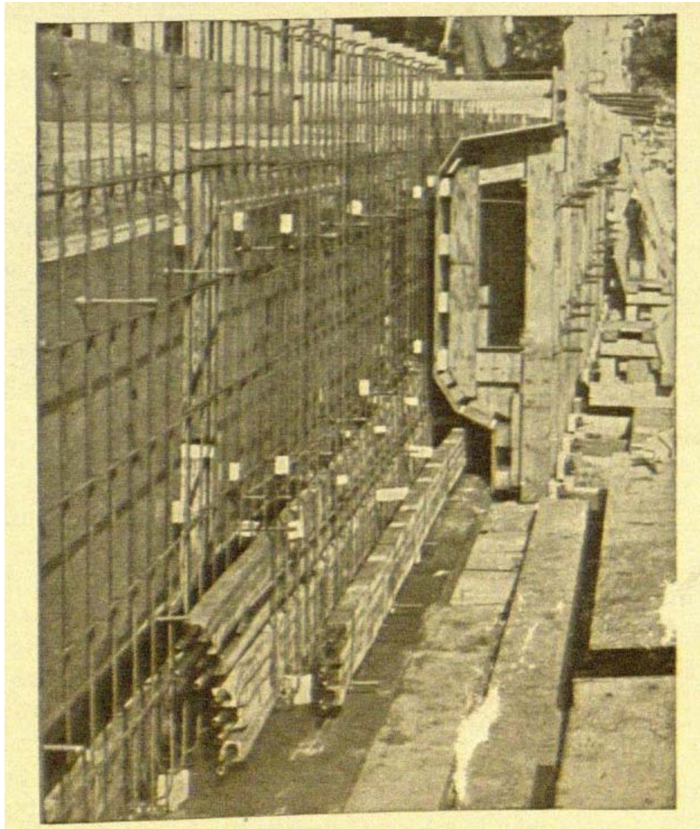

Figure 2-21 Walnut Lane Bridge: Mild steel reinforcement and rubber sheaths in main-span beam (Magnel 1954).

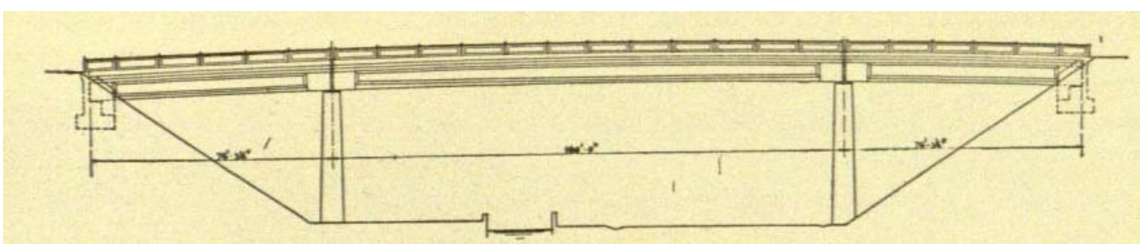

Figure 2-22 Walnut Lane Bridge Elevation (Magnel 1954)

2.3 Urlich Finsterwalder

Urlich Finsterwalder, Figure 2-23, just like Eugene Freyssinet, started out his career as a builder, and many of his designs were only considered because his new methods helped him to out-bid competitors (Billington 2004).

Figure 2-23 Urlich Finsterwalder (Billington 2004)

Finsterwalder, much like Gustave Magnel and Eugene Freyssinet, was exposed to prestressed concrete after having much experience in traditional reinforced concrete. Like Freyssinet, Finsterwalder specialized in arch and thin shell structures in reinforced concrete.

He also was in the construction and design industry during World War I and World War II. The wars deeply affected many of the major designers of this period including Finsterwalder. For example, some designers were not allowed to have any contact with designers from enemy countries, while others were able to use their new methods and systems for war time building and rebuilding. Others, such as Urlich Finsterwalder, made the best of a horrible situation and turned their experiences during the war into a learning experience. "Finsterwalder learned mathematics while in a French prison camp during World War I (Billington 2004)," allowing him later to put his ideas into practical engineering, and enabling him to much more easily communicate his ideas and methods to other bridge designers and constructors.

After WWI, Finsterwalder put this newfound knowledge to good use. His theory served as the basis for many terrific thin shelled concrete structures which were designed and built by Dyckerhoff and Widmann A.G., starting in the mid-1920's. After WWII he began his major prestressing work when he developed the "Dywidag System" for prestressing, which was described earlier in section 2.2.1.7.1.12. He used this system in many of his bridge designs including the "Neckar Bridge" to connect the districts of Ludwigsburg and Waiblingen near Stuttgart. Finsterwalder describes this design procedure very well in his report in the Journal of the American Concrete Institute (Finsterwalder 1952).

Ulrich Finsterwalder sought to show that prestressed concrete could directly compete with structural steel as a building material, not only economically but also with respect to design capacity for long spans with minimal depth. In David Billington's words (2004), "Finsterwalder showed that prestressed concrete can be a safe, economical, and elegant solution to almost any major structural problem that exists in the modern world." The following design and construction methods were Finsterwalder's way of proving that prestressed concrete could compete directly with structural steel.

2.3.1 Double Cantilever Method of Bridge Construction

Urlich Finsterwalder's major bridge idea is the double cantilever design method (Figure 2-24), which he developed right after World War II, using prestressed concrete as the major structural material. The major advantage of this construction technique over others of the time was that these bridges were constructed entirely without scaffolding, reducing a significant cost of the construction of a bridge.

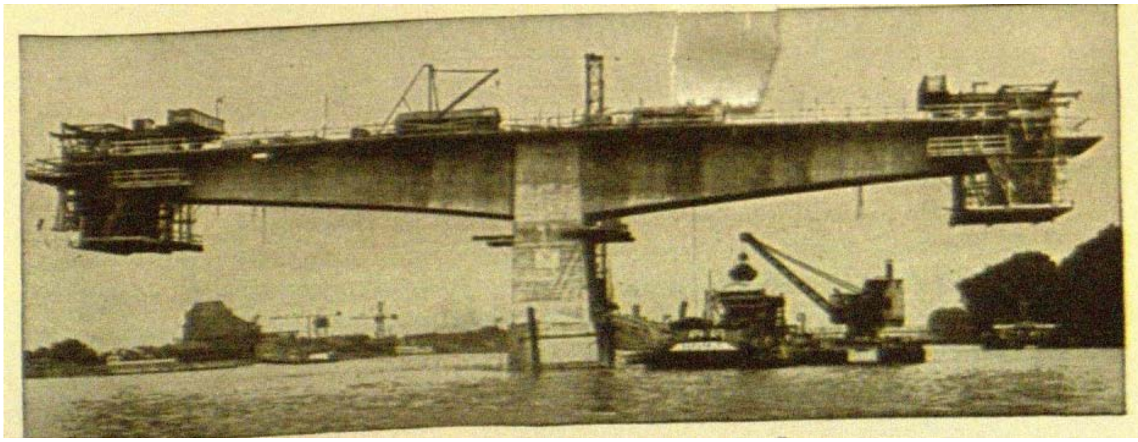

Figure 2-24 Double Cantilever Method (Magnel 1954)

Urlich Finsterwalder describes his double cantilever method of design (1965): "The free cantilever system of prestressed bridge construction was first applied in 1950 to a bridge across the Lahn River at Baulduinstein, Germany. Two years later the method received worldwide attention with the construction of the bridge over the Rhine River at Worms. This bridge has a main span of 370 ft (113m)." After this until the time that the article was written in 1965, 86 bridges were constructed using the free cantilever system.

2.3.1.1 Bendorf Bridge

The Bendorf Bridge, completed in 1964 and, shown in Figure 2-25 is a concrete box girder bridge that spans the Rhine River at Bendorf near Koblenz, Germany. It spans 682 feet (208m) which made it the longest spanning box girder bridge at the time of Finsterwalder's article. The design of this bridge, including its material and shape was subjected to a public competition to see not only who could come up with the most aesthetically pleasing bridge, but also the most economical bridge design. One difficulty for many designers of short span bridges was that the river channel had to remain navigable to heavy barge and boat traffic throughout the construction. Thus, the channel was required to maintain a width of 328 feet (100) during erection and hold a final navigable span of 672 feet (205m). Consequently, Urlich Finsterwalder's cantilever system was selected for the central span of the box girder bridge. This system utilized a hinge at the midpoint designed to transmit only shear forces. This hinge point made possible a monolithic cast of the box girders with their main piers avoiding very costly steel shoring and allowing the river to remain navigable. Figure 2-25 shows the Bendorf Bridge close to completion.

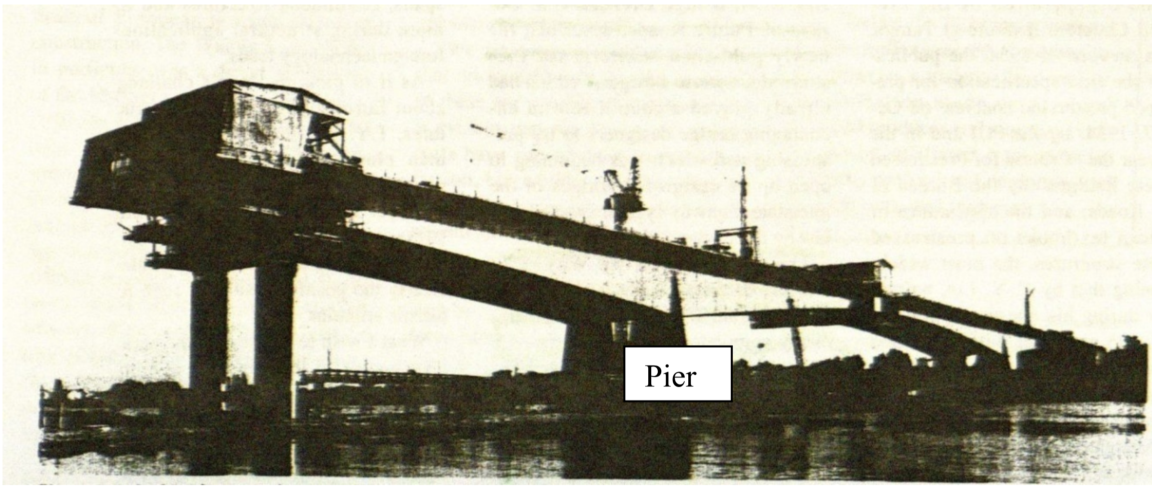

Figure 2-25 Bendorf Bridge over the River Rhine, Germany (Billington 2004)

2.3.1.1.1 Structural Details of the Bendorf Bridge

The Bendorf Bridge in Figure 2-24 consists of twin, independent box girders continuous over seven spans with an overall length of 1650 ft (503m). "The cross section consists of two monocellular hollow boxes with a combined width of about 100 feet (30.5m). Each hollow box consists of a road slab 43 ft (13m) wide, two 1 ft (30cm) thick longitudinal webs, and a bottom slab 24 ft (7.3m) wide. The box depth varies from 34 feet (102m) at the main piers to 14 ft (4.3m) at the center of the bridge giving depth-to-span ratios of 1/20 and 1/50, respectively (Finsterwalder 1965) ."

The slab is 17 inches (43cm) thick at the central piers and 11 inches (28cm) thick at mid-span. The bottom slab is 8 ft (2.4m) thick at the piers and 6 inches (15cm) at mid-span. The central piers are only 9 feet (2.8m) thick and have a foundation depth of 52 feet (15.8) below the riverbed as shown in Figure 2-25. The piers have caissons, which are 23 feet (7m) wide and 110 ft (33.5m) long (Finsterwalder 1965). The designer chose not to use symmetry in his design because the sidewalks and terrain have nonsymmetrical features. Subsequently, "The superstructure alone required 2800 sheets of analysis that produced 160 drawings (Finsterwalder 1965)."

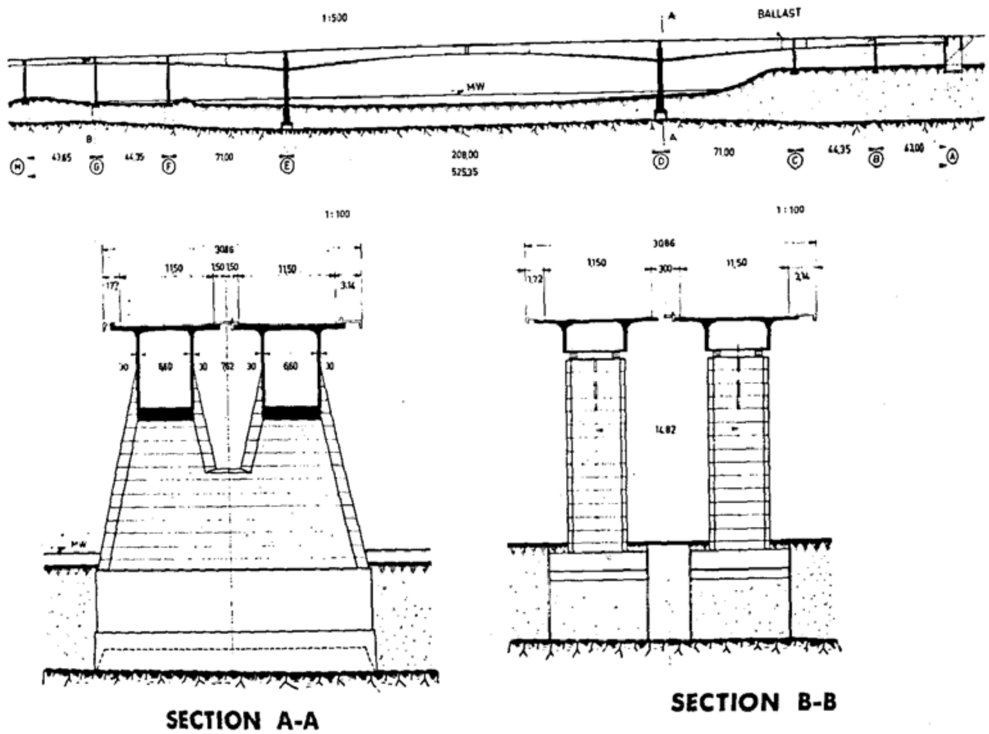

Figure 2-26 Longitudinal span and cross sections of Bendorf Bridge (Finsterwalder 1965)

2.3.1.1.2 Prestressing of the Bendorf Bridge

The Bendorf Bridge is prestressed in three ways: Longitudinally over the entire cross-section, transversely in the deck, and inclined in the webs. Five hundred and sixty, 1 ¼" (6mm), high-grade steel reinforcing bars were uniformly distributed over the two main piers in the deck to handle the negative moment from the cantilevers. The designer only allowed the compressive stress in the bottom slab to reach 1800 psi (12.4 kPa). While the concrete of the compression slab at the central part of the cantilever arm was heavily reinforced to reduce the dead weight of the bridge. This dramatically decreased the thickness of the slab. Furthermore, due to the prestressing in the bridge, the overall tensile stresses in the concrete were negligible.

Figure 2-27 shows how the longitudinal prestressing decreases in uniform steps from the main piers to the central hinge and to the adjoining piers. Due to this decrease, the shear forces in the hollow box webs on both sides of the piers are almost constant, which renders, the inclined prestressing and shear prestressing, nearly constant throughout the cross—section.

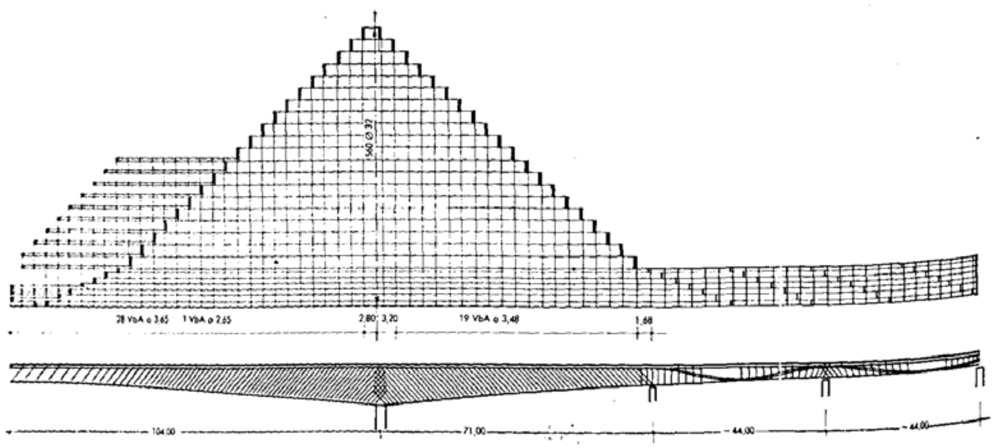

Figure 2-27 Section showing prestressing tendons in webs (Finsterwalder 1965)

The Dywidag method of prestressing, developed by Finsterwalder, was used on this bridge due to the simple connections of the threaded bars. This greatly simplified the step-by-step construction. The details of this system were described in section 2.2.1.7.1.12.

2.3.1.1.3 Construction of the Bendorf Bridge

The construction of the Bendorf Bridge started on March 1, 1962. Once the pier foundations were completed, the free cantilevering operations began starting with the west river pier (Figure 2-28). The cantilever construction started in July, 1963 and was already completed by the end of 1963. Clearly, this is a very quick and cost effective construction method because no scaffolding means much quicker construction and, as stated earlier, significantly reduced cost.

"Moving forms were used to construct a 12 foot long section each week. After casting, stripping, and post-tensioning, the forms were moved farther away from the pier for the next 12-ft section. The one section per week schedule per side was increased to two sections as they became shallower. The moving formwork structure was enclosed during cold weather to provide progress through the winter (Finsterwalder 1965)."

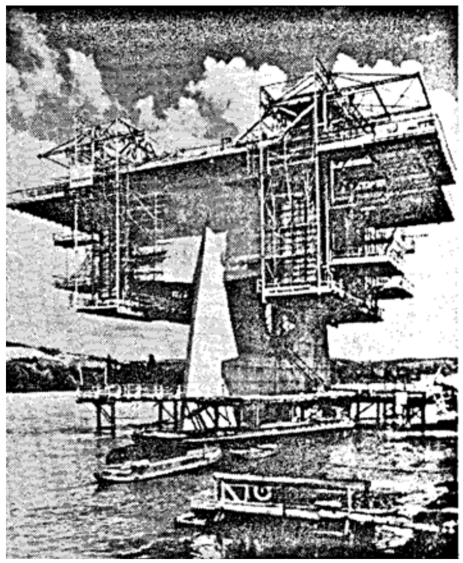

Figure 2-28 Cantilever operation on the west river pier (Finsterwalder 1965)

During the construction, much care was taken with alignment to achieve the shape of the bridge exactly as designed, particularly difficult since this bridge consisted of numerous individual sections. In the middle of the river, the deformations due to creep and shrinkage amounted to around 10 inch (25.4cm) deflections. Also, full traffic loading produced additional eight inch deflections at mid-span.

The free cantilever method of construction is extremely advantageous in areas with difficult accessibility and is still used today. Specifically, this is many times the solution to mountain crossing bridges because pier heights of up to 300 feet (91.5m) are possible. This method is also an ideal solution for elevated highways which still have to be used in cities. This is depicted in Figure 2-29 at Shibuya, an elevated highway in Tokyo.

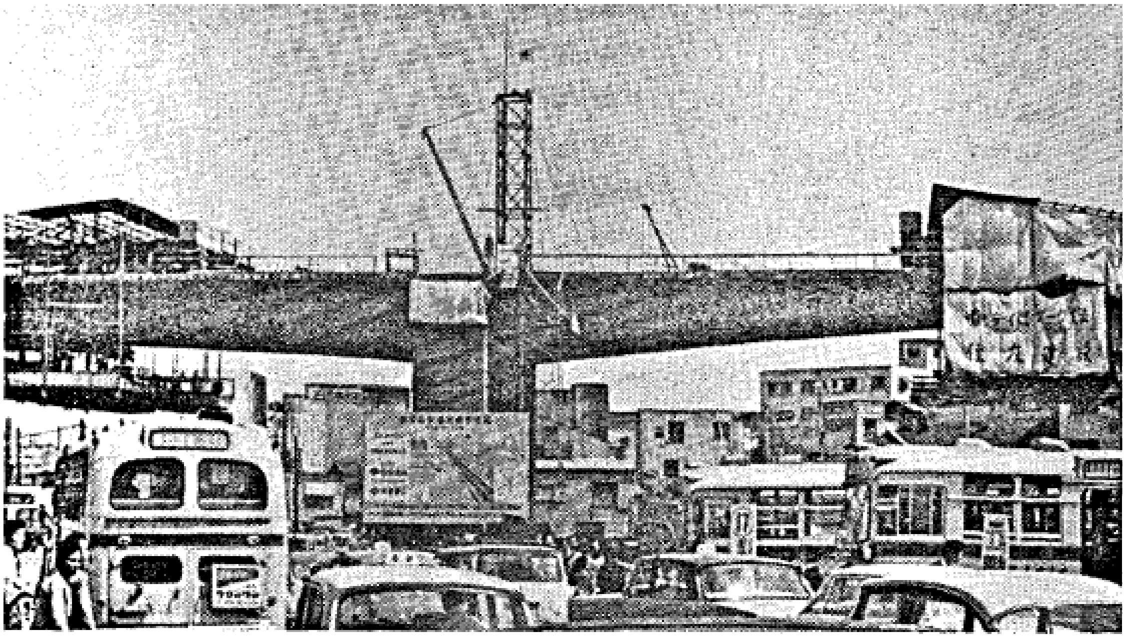

Figure 2-29 Double Cantilever Method: Shibuya elevated highway (Tokyo) (Finsterwalder 1965)

2.3.2 Stress Ribbon Bridge

Finsterwalder, as stated earlier, strove to provide a prestressed concrete solution for every steel bridge design. He believed that prestressed concrete bridges spans could rival the longest spans in steel design. Such long spans previously had been the sole province of steel suspension bridges. However, in the late 1940's and early 1950's, Finsterwalder developed a new concept in prestressed concrete bridge design: Stress-ribbon Bridge.

At this point in history, the stress ribbon bridge was a theoretical idea. It had not yet been constructed. The first public use stress-ribbon bridge was built in Switzerland in 1965. Stress-ribbon bridges are primarily used for pedestrian bridges with minimal loading.

The basic concept of this design method is a stress ribbon of reinforced concrete, hanging in a funicular curve, anchored in riverbanks. Finsterwalder first proposed this system of bridge design to the city of Geneva for a bridge over Lake Geneva. This bridge holds central and end spans of 1500 ft (457m) long and alternate with 650 ft (198m) spans over the supports (See Figure 2-30). The anchorage structures that resist horizontal thrust were to be located in the banks of the lake (1965). Two other Finsterwaler bridge designs show his proficiency. Figure 2-31 is Finsterwalder's proposed design, for the Bosphorus Strait between Asia and Europe in Istanbul Turkey. Instead, the Bosphorus Bridge was built as a steel suspension bridge in 1973. Also, figure 2-32 depicts Finsterwalder's proposal for the Naruto Bridge in Japan.

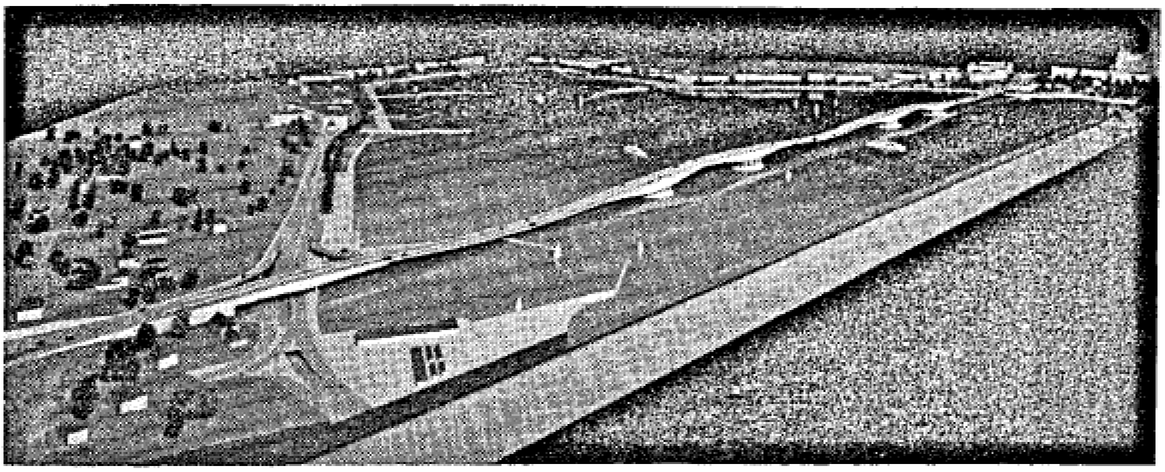

Figure 2-30 Model of Proposed Stress Ribbon Bridge at Lake Geneva (Finsterwalder 1965)

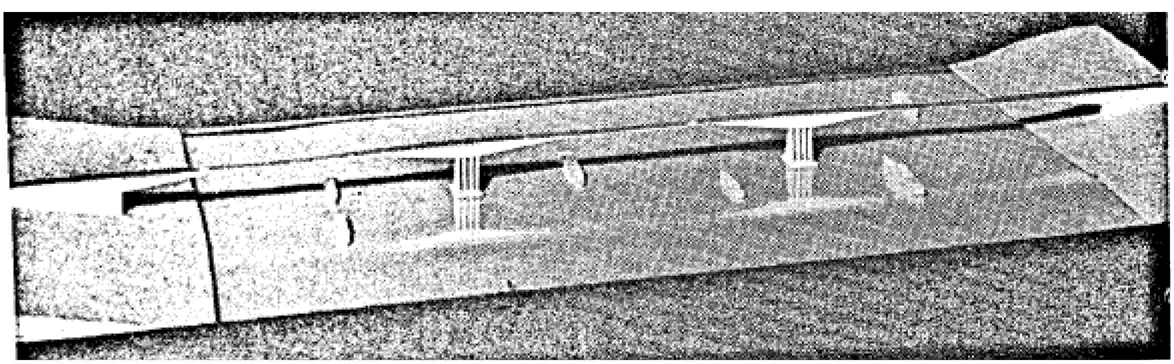

Figure 2-31 Model of Bosphorus Bridge (Finsterwalder 1965)

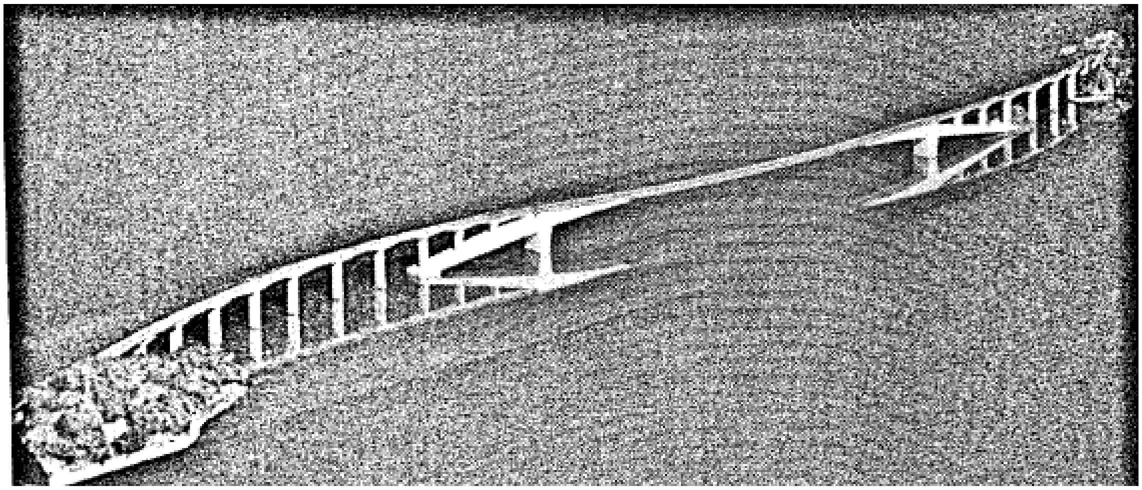

Figure 2-32 Model of Naruto Bridge (Finsterwalder 1965)

2.3.2.1 Finsterwalder's Stress Ribbon Bridge Theory

The stress ribbon bridge combines a suspended concave span and a supported convex span. The concave span utilizes a radius of about 8200 ft while the convex span, depending on the design speed of the bridge, utilizes an approximate radius of 9800 ft (1965).

The stress ribbon itself is a reinforced concrete slab with a thickness of about 10 inches (25.4cm). This reinforcement consists of three to four layers of 1 inch (2.5cm) to 1 ¼ inch (1.2cm) diameter, high strength steel. The layers are spaced so that the prestressing pipe sleeve couplings can be used as spacers both vertically and horizontally. To resist bending moments from traffic, the slab is heavily reinforced at the top and bottom in the transverse direction.

The high strength steel tendons are stressed piece by piece during erection to produce the desired upward deflection radius of 8200 feet (2500m) under dead load of the superstructure plus the pavement. A temporary catwalk is provided to stress the first tendons. The formwork for the bridge is hung from the tendons and then removed once the concrete is cured. Concrete is placed from the middle of the freely hanging suspended concave part and continues without interruption to the supports (Finsterwalder 1965).

2.4 "Partial Prestressing"

Up to this point, 1939, Eugene Freyssinet, among others, believed that two different concrete building materials existed: ordinary reinforced concrete, with mild reinforcing steel, and prestressed concrete. Freyssinet was adamant that prestressed concrete should have no flexural tensile stresses under service loads and that prestressed reinforcement must supply all of the flexural capacity. This allowed for no flexural cracking because the concrete never was allowed to go into tension.

In 1939, a different idea appeared in a proposal by Austrian H. von Emperger (Bennett 1984). Emperger proposed that, along with mild reinforcing steel in ordinary reinforced concrete, a small number of prestressed high strength steel wires should be added. His proposal was not based on crack control, which he considered a very favorable advantage, but to increase the allowable service load by reducing the effective stress in the reinforcement (Bennett 1984). Adding a small prestressed force in an ordinary reinforced concrete beam was able to reduce this, he proposed, by reducing the effective stress in the mild reinforcement. Emperger was the first to propose this concept of partial prestressing, defined as concrete with both ordinary flexural reinforcing and high-strength prestressing tendons. He supported this concept with a series of tests in which 42 percent of the mild reinforcement was replaced by wires that held a very low prestress force. Just one year later, Paul W. Abeles, a student of Emperger, reinforced this concept in his paper "Saving Reinforcement by Prestressing."

2.4.1 Paul W. Abeles

H. Von Emperger coined the idea of partial prestressed concrete, but Paul W. Abeles would develop the concept of partial prestressing into common practice. Abeles commented on partial prestressing in the following statement:

"It is a simplification and improvement to tension only a part of the reinforcement, consisting of thin wires, and to abandon the idea of having a homogeneous building material….until the final stage (Bennett 1984)."

The idea of a homogeneous building material was that of Freyssinet's. His thought was that in prestressed concrete the prestressed wires were not used as a reinforcement steel to provide flexural resistance, such as mild reinforcement in ordinary reinforced concrete. Instead, the prestressed wires were only used to develop enough pre-compression force in concrete so the concrete would never be subjected to tensile forces. He also believed that in ordinary reinforced concrete, the reinforcing steel provided the needed resistance to flexural tensile stresses. He was correct on both counts. The third belief, and his one misconception about these building materials, was that they were two distinctly separate building materials. He thought that each functioned well alone, but that they had no application together. Still in 1949 he boldly stated, at the Institute of Civil Engineers in London (Bennett 1984):

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

Abeles recognized that in partially prestressed members, the stress, below a certain level of loading, would still be entirely compressive. This would keep the concrete from cracking, and it would retain the advantage of being a homogeneous uncracked material. Abeles states, "the compressive stresses in the concrete tensile zone and the unstretched reinforcement…are reduced to zero if only that part of the total load acts which corresponds to the ratio chosen for the prestressing" (Bennett 1984).

Abeles also proposed using, instead of mild reinforcement, high strength cold drawn wires as non-prestressed reinforcement. He argued that even though the high strength steel wires were more expensive per unit, the amount of reinforcement would be reduced greatly because high strength wires had much greater yield strength than mild reinforcement, and the amount of steel saved would offset the price difference between the two materials. He also proposed prestressing the entire high strength steel wires in a section to a much lower tensile stress.

Paul W. Abeles and H. von Emperger thought of partial prestressing as a way to utilized high strength steel wire in ordinary reinforced concrete. If non-prestressed high strength steel were to be used as reinforcing in concrete, the overall area of reinforcing steel would be substantially less, but the member would have large deflections and detrimental cracks at service loads. Partial prestressing offered a way of using this high strength steel in reinforced concrete structures while improving its behavior at service loads (1984). Abeles was very experienced in testing spun concrete poles and reinforced concrete beams made with high strength steel. This experience led him to believe that partial prestressing could be obtained while keeping the deflection and crack width to within tolerable limits.

2.4.1.1 Proving His Theory

Instantly, Abeles presented his paper about partially prestressed concrete, considerable criticism followed. In fact, Abeles' paper was followed by a lengthy published correspondence including was an argument against partial prestressing. The correspondence stated that partial prestressing would counteract the advantages of prestressed concrete; severe cracking would occur and the economic advantages of prestressed concrete, proposed by Freyssinet and Magnel, would be lost.

This controversy would go on for many years with members of both sides continuing the argument for their views on partial prestressing. One of the main problems with the argument, for either side, was that prestressing was early in its developments, and minimal experiments had occurred, so that neither side had solid proof of correctness. Also, both sides had viable arguments backing their ideas. Furthermore, the economic advantage that Abeles discussed could not be reliably tested because prestressed concrete was still relatively new. Most of the designers of prestressed concrete had their own individual method of prestressing, so one method might have produced very good results with partial prestressing while another might have produced very poor results. Abeles was very aware of these problems at this time, but he still believed in his theory. He knew that the only way to prove his concept viable was to test and experiment with partial prestressing. "It will depend on tests to prove whether my ideas are adequate (Bennet 1984)."

In the early 1940's, during World War II, structural designers were very limited in what they were allowed to experiment on and test. Thus, the developments in prestressed concrete at this time in history were minimal until post-war reconstruction. Abeles, though, continued to advocate partial prestressing. He diversified his testing to a much-needed piece of equipment during wartime, railway sleepers. Railway sleepers, also called ties, are the wood, or now prestressed concrete, members, to which the rail is fastened. He carried out small scale testing of these railway sleepers, which he partially prestressed by only tensioning 40 percent of the wires in the beams (Abeles 1945). The tests proved to be very favorable and in line with Abeles' initial proposals. He subjected the beams to overloading then retracted the loads from the beams. When subject to this type of loading, the beams recovered extremely well from deflections, and the cracks, which developed from overloading, closed to more than acceptable widths. Abeles was able to retest some of these railway sleeper beams after two years' service, and they still exhibited all of the same properties as in initial testing.

Paul Abeles was able to apply his theories of partial prestressing to various projects in the post-war reconstruction period in the late 1940's, because many railway over-line bridges needed additional clearance to accommodate electrification. This was necessary particularly where existing masonry arched bridges did not provide desired clearance and had to be demolished. Otherwise, bridge decks' depths had to be greatly reduced for the new clearance limitations. Abeles was granted the contract to renovate many of these bridges throughout Europe, and he decided to use a system of partial prestressing consisting of a composite solid slab with inverted precast prestressed T-beams. Figures 2-33 and 2-34 (Bennett 1984) show the original masonry arch construction and Abeles' new partial prestressed construction. Abeles was able to convince British Railways that partial prestressing was the answer to rebuilding their bridges. He assured them that it was a very economical method that did not jeopardize the safety of the structures.

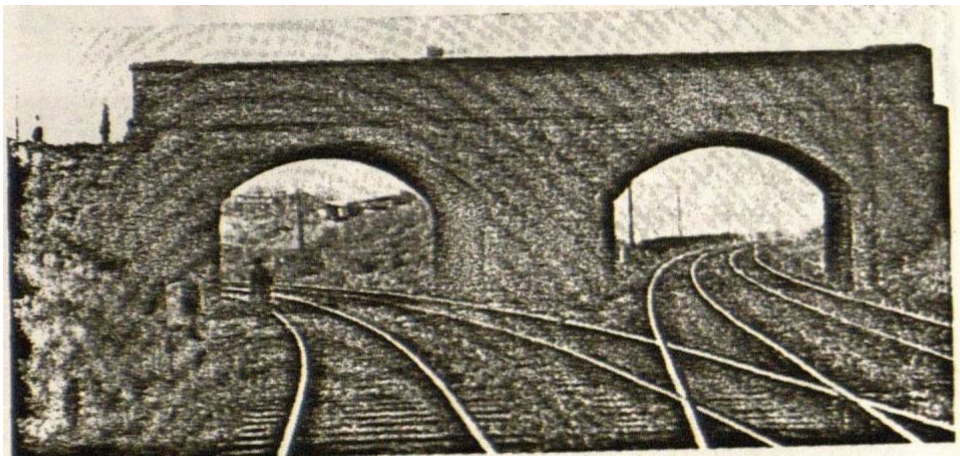

Figure 2-33 Brick Masonry arch bridge before reconstruction (Bennett 1984)

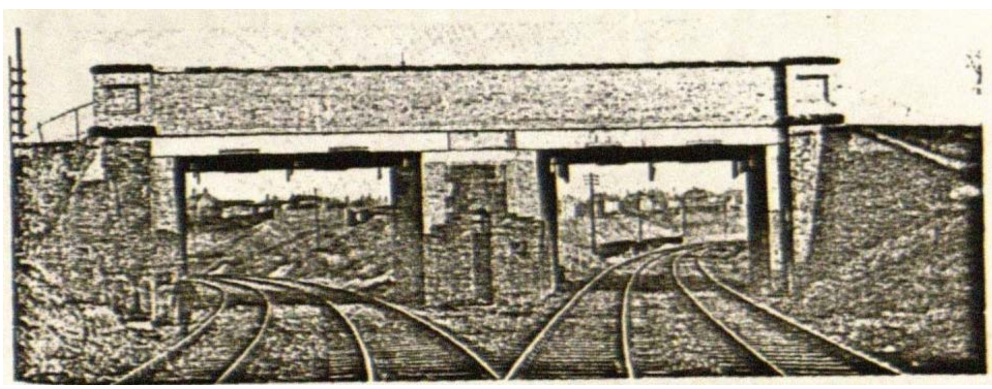

Figure 2-34 Masonry arch bridge after reconstruction using composite partially prestressed concrete deck for overhead electrification (Bennett 1984)

Dr. Abeles, in his first bridge decks, allowed tensile stresses in the concrete of 500 psi at service load (Bennet 1984). At this point, testing had proven that visible cracking did not show up in beams until the tensile stresses reached twice the allowed value, or 1000 psi. To confirm his results with the British Railways, he tested one beam out of each row of beams in the bridges to a tensile stress of 750 (5.1 kPa) to 800 psi (5.5 kPa). Figure 2-35 shows the erection procedure of the inverted T-beams. The tests documented (Bennett

1984) that at these tensile stresses; no cracks were visible throughout the tests. In fact, in one instance, the load was sustained for 30 days during which the deflection increased by 65 percent due to creep, but the beam still did not show significant cracking.

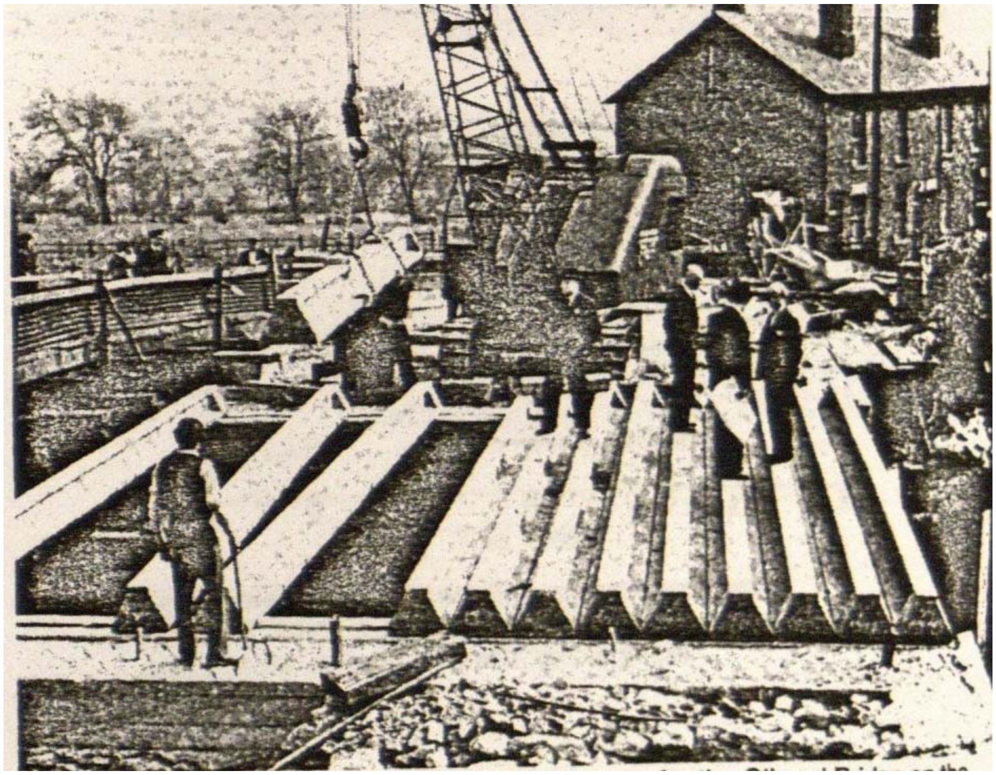

Figure 2-35 Erecting partially prestressed inverted T Beams for Gilyord Bridge on the Manchester-Sheffield Railroad line in 1949. Dr. Paul Ableles is on the right. (Bennett 1984)

Clearly, cracking was not an issue at loading of 1.5 times the service load, but a concern existed about cracking at severe overloading of the structures. Specifically, fatigue in the prestressed wires was a large concern in overloading situations, as it is in all bridge design. Fatigue failure occurs after cyclic loading, many times below design load, over many years. This cyclic loading produces elevated fatigue stresses at or above design loading. Abeles, still confident about partial prestressing, decided to conduct a repeated loading (fatigue) test of his partially prestressed composite bridge deck design. Figure 2-36 shows one of the beams being tested at a precast prestressed concrete plant. He decided to use the same slab that had been previously loaded to cause, in theory, flexural cracking.

“The previously loaded slab was subjected to one million cycles of a load at which the stress in the concrete before cracking of the slab would have varied from 102 psi compression to 553 psi tension; for the second million cycles the maximum stress was increased to 800 psi, and for the third million cycles, the range of stress was from 436-902 psi (Bennett 1984).”

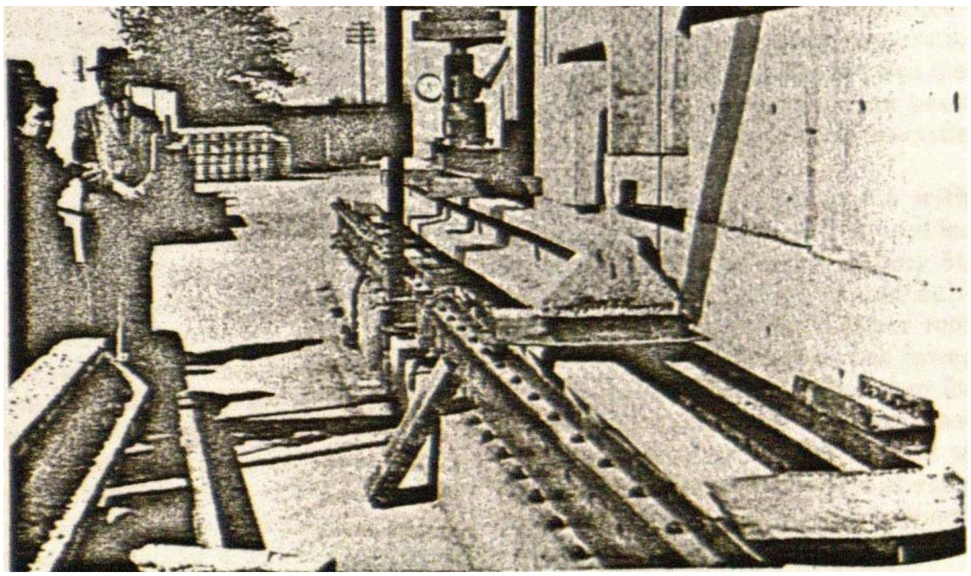

Figure 2-36 Fatigue test of partially prestressed concrete inverted T Beam at precast prestressed concrete plant (Bennett 1984)

Each repetition of loading, up to two million cycles, produced visible cracking of the slab at maximum load, but the visible cracks disappeared when the load was removed. After the beam was subjected to the third million cycles, the cracks, after loading was removed, returned to a state of “only just visible” as Bennett (1984) describes. After three million loading cycles, the beam was loaded to failure, whereupon, it failed at approximately the same ultimate load as the same slab would have had it not been fatigued.

In this composite bridge-slab design, Abeles included mild steel reinforcement in cast-in-place concrete along with his inverted, prestressed T-beams. Figure 2-37 shows a cross-section of Abeles’ design. This figure shows a blow—up of the cross-section of one of the beams, showing the prestressing steel as well as the mild reinforcing. Also, in the other section, the mild reinforcing is shown in the composite beam. After Abeles completed his tests, his original design was put into effect. He used high strength prestressing wires as non-prestressed reinforcement in the inverted T-beams as well as mild reinforcement in the cast-in-place concrete. The un-tensioned wires were placed in pairs at the bottom flange of the beams seen in Figure 2-38. The addition of high strength, non-prestressed wires reduced the amount of site work and required only about one-fifth of the amount of mild steel that would have been necessary. Under ultimate load conditions, the stress developed in the wires was shown to be almost equal to their tensile strength (Bennett 1984).

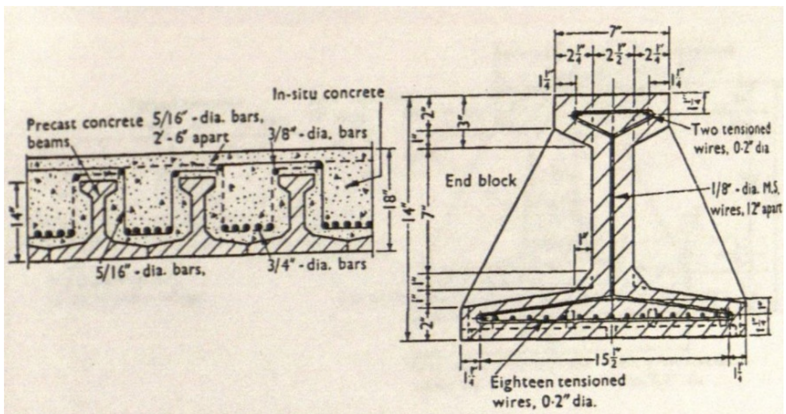

Figure 2-37 Composite partially prestressed bridge deck with non-prestressed reinforcement (Bennett 1984)

Abeles also designed precast beams used as roof beams. Since the flexural load was not as significant as the bridge beams, so he was able to lower the number of pretensioned wires and raise the number of untensioned wires. These designs were first used in the roof of a freight depot at Bury St. Edmunds, England in 1952 (Fig. 2-38) (Bennett 1984). Roof beams for a locomotive depot in Ipswich, England also utilized this partial prestressing method.

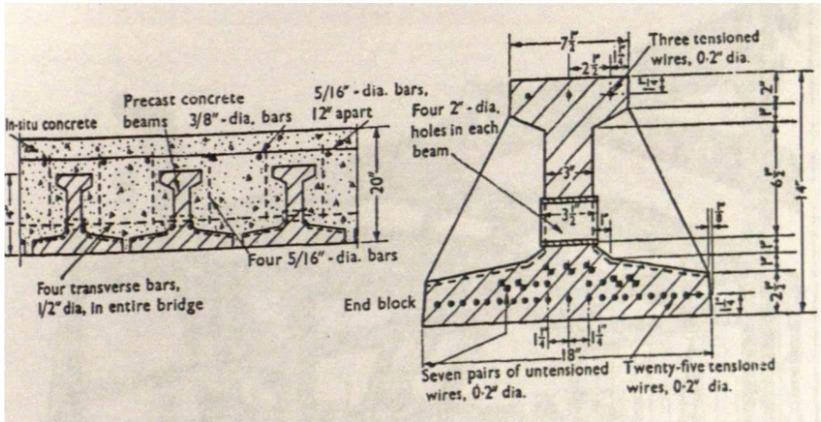

Figure 2-38 Bridge deck with non-prestressed reinforcement of high strength steel wire (Bennett 1984)

After seeing the success that Dr. Abeles was having with his "partially prestressed" concrete, Freyssinet finally modified his position on this subject. He accepted that tensile stresses of around 725 psi (5 kPa) might, and "indeed should" (Bennett 1984) be permitted in a bridge. Bennett also goes on to say, "…his [Freyssinet's] achievements and prestige at this period were so great that statements such as the one quoted were bound to create early difficulties for the development of partial prestressing." This simply suggests that many designers who had listened to the first bold statements of Freyssinet on full prestressing or no prestressing at all, totally disregarding his modified position on partial prestressing.

Today, most prestressed and post-tensioned concrete in structures utilizes Abeles' partial prestressing. However, it is no longer called 'partial prestressing,' mainly because almost all prestressed/post-tensioned concrete has mild reinforcement. Also, by code, prestressed concrete is allowed to transition into the tension region in three stages as defined by ACI. The first stage is uncracked, then the prestressed concrete goes into a transition stage, and finally it is designated as a cracked material (ACI 2005)

CHAPTER 3 - Prestressed Concrete in the United States

After the Walnut Lane Bridge was designed by Gustave Magnel and built in Philadelphia, prestressed concrete quickly caught on in many parts of the country, designers started to invent their own methods of prestressing to get a jump on the market and quickly patent their own designs. While prestressed concrete would have without a doubt caught on in the United States, several individuals as well as several companies pushed this idea. They believed that prestressed concrete was the building material of the future and did everything in their power to develop it as quickly as possible.

3.1 The Roebling Family Tradition

The Roebling name is very familiar to many of today's structural engineers, especially those designing bridges. This name brings to mind the legendary John A. Roebling who founded John A. Roebling and Sons Company.

John A. Roebling was granted the honor of designing the famous Brooklyn Bridge, a 1600 foot (488m) steel suspension bridge. At the time of its construction, this bridge became the longest spanning suspension bridge by about twice the previous span length. Roebling, for the first time on the Brooklyn Bridge, used high strength steel wire rope, with an ultimate strength of 160,000 psi (1103 mPa). For the previous structures, which held the longest span, and which Roebling also designed, wrought iron cables had been used with about half of the strength of the new high strength wire (Zollman 1980, p. 137).

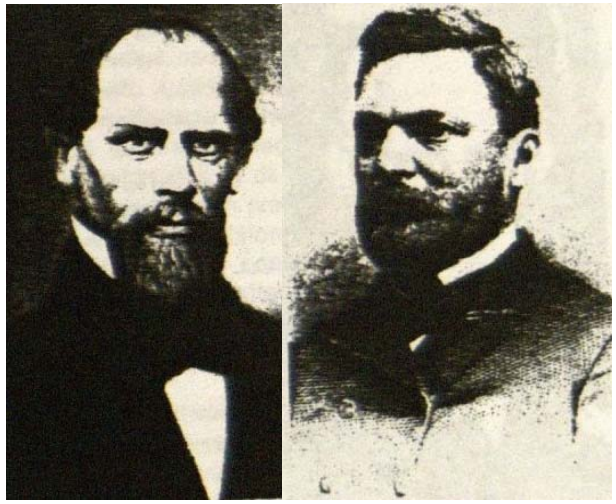

Figure 3-1 John A. Roebling and Washington A. Roebling (Zollman 1980)

As stated by Charles Zollman (1980), "The Brooklyn Bridge would serve as a model for such titans as the George Washington Bridge, the Golden Gate Bridge and the Verrazano-Narrows Bridge, the latter having a span from tower to tower of 4260 ft."

3.1.1 Charles C. Sunderland

Charles C. Sunderland was a chief bridge engineer at Roebling for many years; he died in 1952. Sunderland was basically a structural steel oriented bridge engineer, but immediately after Freyssinet introduced prestressed concrete to the world in the late 1930's, Sunderland became interested (Zollman 1980). In particular, in 1944, L. Coff, in New York, had caught Charles C. Sunderland's attention by describing the European developments in prestressed concrete. Sunderland was convinced both of the potential of prestressed concrete in the United States and that high strength steel had its place with prestressed concrete.

Sunderland quickly worked at convincing the management at John A. Roebling and Sons that a sizable amount of money should be invested into researching prestressed concrete design and construction. This would lead to the development of technical know-how on job sites as well as to the development of materials and equipment specially designed for prestressed concrete construction. Sunderland worked very closely with the research and development teams at Roebling casting and testing members. This work finally produced the American post-tensioning system known as the Roebling post-tensioning system (Zollman 1980, p. 139).

Sunderland continued this research and development and by 1945 he felt that Roebling was ready to manufacture the steel components for the Roebling Post-tensioning system; however, he made sure that the research continued. In a 1945 report, Sunderland looked ahead in to the future of prestressed concrete. When others were just learning of this new prestressed concrete concept, Charles Sunderland was already predicting that it would become standard practice to use prestressed concrete as the main building material in single and multiple story buildings as well as bridges, airport runway slabs, and highways.

In 1951, "Roebling – Strands and Fittings for Prestressed Concrete" prestressed concrete materials catalogue was published in America (Zollman 1980). The first of its type ever published. In 1955, an updated version, "Roebling – Tensioning Materials for Prestressed Concrete," was offered to the rising industry of prestressed concrete. This gave designers a choice of anchorage systems and strands to use in their designs. Before this publication, such devices were very hard to obtain, very expensive, and almost impossible to customize specialized anchorage devices for only one project. For example, one of the major developments in prestressed concrete was stress-relieved wire that is now available to all designers.

3.1.1.1 Stress-Relieved Wire

L. Coff, under the direction of Charles Sunderland, submitted a preliminary design for the Walnut Lane Bridge in Philadelphia. Even though Gustave Magnel ended up winning the bid for the design and construction of this project, Roebling and Sons was not far behind. Sunderland gracefully accepted the rejection, and instead of dwelling on it, he said, "Well, we shall now proceed with the manufacture of a cold drawn wire with qualities second to none. (Zollman 1980, p. 140)"

He did, in fact, create this 'ultimate' material. He produced the stress-relieved, 0.276-in diameter high-strength cold drawn wire not long after this comment. According to Zollman (1980), the Walnut Lane Bridge was the first structure in the world to use this high quality, high strength, and stress-relieved wire. Even Magnel commented (Zollman 140), "Had I known that this kind of wire was available in the United States, I would have specified a much smaller number of wires for the Walnut Lane Bridge."

Many other structures were built with the stress-relieved wire, but with the advent of the 7-wire strand, use of the 0.276-in stress-relieved wire gradually ceased.

3.1.1.2 Stress-Relieved Strand

Sunderland was never totally happy with a single product that he developed because in his eyes, there was always room for improvement. After creating stress-relieved wire, he tried to develop a stress-relieved strand. His first attempt at this was a 5/16-inch strand made out of stress-relieved wire. "This did not work because cold-forming the outside wires around the center wire destroyed most of the benefits of stress-relieving (Zollman 1980, p. 142)."

A stumped Sunderland turned to Roebling's chief metallurgist, Howard J. Godfrey, known as Hank, who first made the strand from as-drawn wires, and then stress-relieved it.

A quote from Charles Zollman (1980, p. 142) about the accomplishments of Charles C. Sunderland wraps up the quality of person that Sunderland was: "The ultimate measure of a man is where he stands in times of challenge and controversy. Charles C. Sunderland, a great and dignified engineer and a true leader of men, stood for progress and growth in the midst of the challenges and controversies of the fledgling prestressed concrete industry. It was Sunderland who taught prestressed concrete to such men as Kent Preston, Lloyd Hill and Pat Patterson, who subsequently made important contributions to the industry." In turn, these latter three sought to expose as many people as possible to prestressed concrete and worked hard to advertise prestressed in magazines such as *Engineering News Record* (ENR), *Architectural Record, Concrete* magazine, and *PCI Journal.* They also wrote many technical articles on design, construction, and cost of prestressed concrete.

Preston, Hill, and Patterson worked for Roebling and advocated for more research and development; in particular, they tried to establish a market for wire products in the United States for prestressed concrete. Again, Zollman (1980) sums up the accomplishments of these Three Musketeers, "Inspired by the great Roebling tradition of quality, the three musketeers, Preston, Hill, and Patterson undertook to educate and assist, to advise and encourage those Americans who, with vision, courage and imagination, ventured in the arena of prestressed concrete construction."

3.2 1950: The Beginning of a New Realm in Prestressed/Post-Tensioned Concrete.

Starting in the 1950's, after the completion of the Walnut Lane Bridge, construction in the United States expanded extremely quickly. This was true for prestressed concrete as well as for all other building materials. After Roebling and Sons invented the stress-relieved strand, designers quickly developed their own anchorage devices for this versatile reinforcing material. At this point, no one standard anchorage device existed. The European button-headed tendon was quickly taking over as the standard, but had not yet been exclusively implemented because designers were still trying to invent their own techniques and methods. At this point, the industry expanded so rapidly in many parts of the world that only a brief description of some of the more important landmarks of prestressing will be described in this report due to scope. The Prestressed Concrete Institute (PCI) Journal recorded a series titled "Reflections on the Beginnings of Prestressed Concrete in America". This series describes in-depth many events that happened in the U.S. in this period of design in the United States, several of which follow:

In 1950, California would have the West's first prestressed pedestrian bridge. This was a particularly important bridge because it proved that prestressed concrete could be used effectively in high seismic regions. The Arroyo Seco Pedestrian Bridge utilized the headed wire method of post-tensioning as shown in Figure 3-2. This is also called the button headed tendon or the Swiss "BBRV." Ken Bondy gave a very thorough description of this:

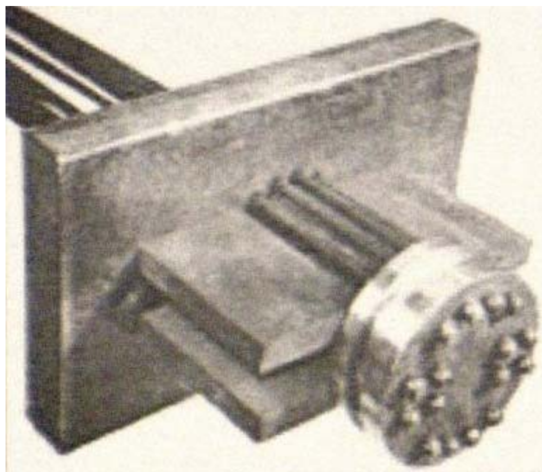

Figure 3-2 Button Headed (BBRV) Anchorage (Bondy 2006)

"A button-headed tendon has parallel, ¼ inch-diameter cold-drawn wires, each with about a 7-kip (7000-pound) effective force, generally six or seven wires per tendon. To secure the wires at each end, they were passed through round holes in a rectangular steel bearing plate and a circular stressing washer, usually externally threaded. Then a "button" was formed on each end of the wire by dynamic impact—basically hammering the steel end of the tendon. The buttons, too big to pass back through the holes, could then be anchored against the stressing washer. A mastic coating was applied to the wires for corrosion protection, and they were wrapped in heavy waxed paper to prevent bond with the concrete. All of this was done in the shop, and then these tendon assemblies were transported to the job. Tendon assemblies were installed into the forms, and the concrete was placed. When the concrete reached a minimum strength, the tendons were stressed to the required tension and elongation with a hydraulic jack attached to the threaded stressing washer. A steel shim exactly as long as the calculated elongation then was inserted between the bearing plate and the stressing washer to hold the elongation and stress in the wires. There was no room for error—the length of the wires and shims had to be exactly predetermined (McCraven 2001)."

Also in 1950, lift-slab construction turned to the prestressed industry. In lift-slab construction, depicted in Figure 3-3, the floor slabs of the building were all placed at ground level and then hydraulically jacked to their desired elevations once the concrete had cured. In an interview, Ken Bondy answered the question, "How were lift-slabs constructed (before prestressing)?" He stated, "Originally in lift-slab buildings, the concrete floor slabs were reinforced with mild steel. The slabs were precast on the ground in a stack and then lifted individually into position using hydraulic jacks at the tops of the columns. While this was an inherently efficient process, there were two problems. First, the slabs tended to stick together as they were lifted, their weight causing them to crack as they were pulled apart. Second, since spans of 28-30 feet were common, and the slabs were 10-12 inches thick, deflection was a serious problem. Midspan deflections of 2 to 3 inches and partition cracking were common in early lift-slab construction." Lift slab designers turned to prestressed concrete designers to solve this problem. Using prestressed concrete, namely cast-in-place post-tensioned systems, effectively reduced the slab thickness and controlled the deflections very efficiently (McCraven 2001).

Thin shelled structure designers, especially folded plate designers, looked towards prestressed concrete as a solution to meet the needs of very creative architects of this period. Many of the most architecturally driven concrete structures come in the form of thin-shelled structures. These designers would have never been capable of keeping up with the evolution of architecturally driven projects without the implementation of prestressing.

Christian Menn, a Swiss engineer and builder, used prestressed concrete to design some of today's most elegant bridges, such as the Charles River Bridge in Boston, MA. He has won numerous awards in European bridge competitions in the past years. Swiss engineers have a long tradition of very elegant long-span bridges and Menn utilized prestressed concrete to continue the Swiss legacy in bridge construction.

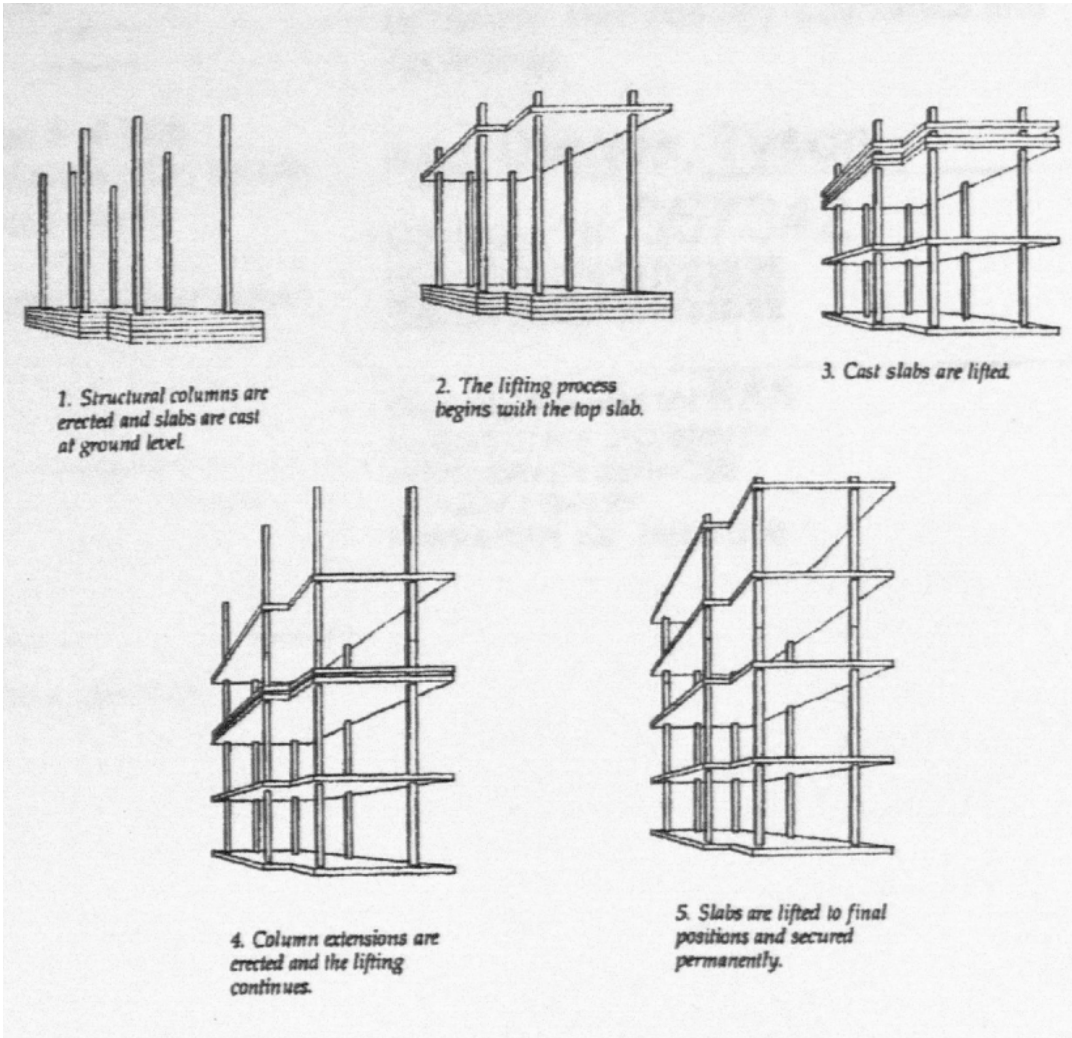

Figure 3-3 Typical lift slab lifting sequence (Russillo 1988)

3.2.1 T.Y. Lin

In 1954, T.Y. Lin, after returning from Belgium, started to actively design bridges and buildings in the west coast region. While designing structures, Lin started writing a book on the design of prestressed concrete. Never thinking small scale, in 1956 T.Y. Lin started to organize the First Prestressed Concrete World Conference (Zollman 1980, p.

142) which, in 1957 was a great success with more than 1200 engineers from 30 countries in attendance including for the first time engineers from the Soviet Union.

3.2.1.1 Load Balancing Method (Lin 1963)

The first approach to the design of prestressed concrete members was the elastic design method. This was used by many of the first designers including Eugene Freyssinet and Gustave Magnel. The next design method was the ultimate strength design method. However, in 1963 T.Y. Lin proposed a third design approach, which he titled the Load Balancing Method for design and analysis of prestressed concrete structures (Lin 1963). Figure 3-4 depicts the load balancing of a post-tensioned beam. Before the Load Balancing method, post-tensioned design of indeterminate structures was a very thorough but tedious task. According to Ken Bondy the transition from ultimate strength design to load balancing was heartily endorsed in the structural engineering world.

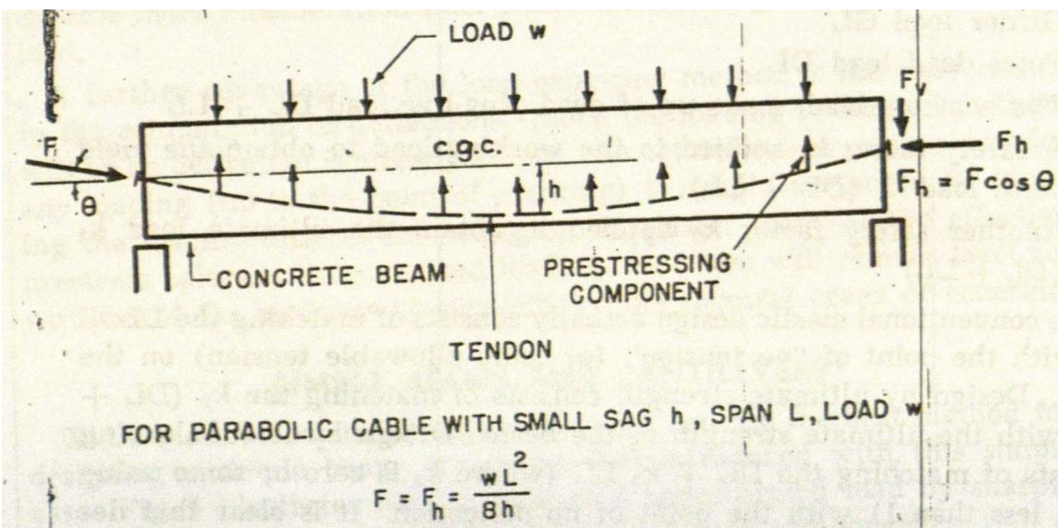

Figure 3-4 Load balancing design of prestressed concrete (Lin 1963)

When asked if post-tensioning design is difficult and tedious, here is how Bondy responded (McCraven 2001):

"Yes, without the benefit of computers, that would be an understatement! Most post-tensioned beams and slabs in building construction are what we structural engineers call "indeterminate;" that is, they have continuous multiple spans and require special techniques for analysis. Prior to 1963, analysis techniques for indeterminate prestressed members were tedious, highly mathematical, and non-intuitive. T.Y. Lin solved this problem for the design engineer. In 1963 in the ACI Journal, he published a revolutionary paper on the analysis of indeterminate prestressed concrete members using a method he called "load balancing." Lin demonstrated how during design, the tendons could be thought of as being replaced by the loads they exert on the concrete member. Once this was done, the structure could be designed like any other non-prestressed structure. Using load balancing, post-tensioned structures could be analyzed fully and accurately using any standard structural engineering technique, such as moment distribution. The introduction of the load-balancing method made the design of indeterminate post-tensioned concrete members about as easy for the practicing engineer as design of non-prestressed members."

Essentially, T.Y. Lin's load balancing method revolutionized the industry in the early 1960's as explained by Bondy (2006). Indeed, post-tensioned concrete construction grew exponentially in the late 1960s and 1970s. Today, we still use T.Y. Lin's load balancing method in our design of prestressed concrete members.

CHAPTER 4 - Conclusion

From 1888 when the concept of prestressing concrete was in its infancy, to the mid 1950's, prestressed concrete evolved from being an unacceptable building material to being a possible solution to almost any structural engineering project. P.H. Jackson developed the first patent of a prestressing application to strengthen a structure, but it was not until Eugene Freyssinet's work that the idea of prestressing could be expanded upon. He was without a doubt the pioneer of prestressed concrete and the discoverer of creep and the need for high strength steel and high strength, high quality concrete.

Eugene Freyssinet was the pioneer of the idea and Gustave Magnel was the pioneer of the science and teaching behind prestressed concrete. He truly revolutionized the idea with his book *Prestressed Concrete.* Magnel displayed that he had a very thorough knowledge of prestressed concrete and that he also was open to new ideas. In just a few years, he released three different editions of his book, which confirmed that he accepted others' ideas and wanted to present them to the structural engineering world. He also printed this book in many different languages, which demonstrated his interest in furthering the knowledge of his peers in engineering.

Urlich Finsterwalder, like Eugene Freyssinet, was a bridge builder. At the time of much of his bridge building, especially in the later years, prestressed concrete was an available building material. However, instead of adjusting his building material to his building technique Finsterwalder, much like Freyssinet, adjusted his building techniques to maximize the possibilities of the available building material. Finsterwalder invented the free-cantilever bridge construction method as well as the idea of the stress ribbon bridge, which revolutionized both prestressed concrete bridge design and construction.

Structural engineers have always aimed to improve upon what's available, seeking in particular to improve on materials and methods in design and construction. The Roebling family started their engineering tradition designing and building projects such as the Brooklyn Bridge. They utilized all available building materials such as cast iron in the early years, structural steel, reinforced concrete, and with the help of Charles Sunderland, prestressed concrete. Sunderland convinced Roebling that it would be in the company's best interest to invest in the research and development of prestressed concrete anchorages and devices. As a result, they finally produced the first ordering catalog for prestressed concrete accessories.

T.Y. Lin in 1963, once again revolutionized prestressed concrete with his new load-balancing method of design. This allowed general practicing engineers to safely, quickly, and easily design prestressed concrete members.

All of the designers and builders mentioned set precedent for engineers and builders in the prestressed concrete world of today. Through their extensive research and development, they helped prestressed concrete transition from not existing in the late 1800s, a very specialized building material used only in short span bridges in the early 1900s, to being the versatile material and method it is today. Specific uses range from small to large projects, from precast-prestressed cattle feeders, stadium risers, and double-T and hollow core slabs, to post-tensioned parking structures and thin shelled structures, all the way to stress ribbon bridges, which compete directly with steel suspension bridges. Prestressed concrete design has grown by leaps and bounds in its' applications in the last 121 years since P.H. Jackson's patent in 1888. It will be interesting and exciting to see what new developments will be discovered and invented in the next 75 years. For instance, new concepts that will without a doubt be integrated into prestressed concrete are synthetic composites including fiber reinforced polymers (FRPs).

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Patents Important to Prestressed Concrete

A.1 The Various Systems of Prestressing for Structures, Strained by Bending.

No.	Purpose of Prestressing	Process Suggested	Name of Proposer	First Publication: Year	First Publication: (Place - Time of Issue)	Pre- Stretching	Post- Stretching	Way of Achievement of the Purpose
1	Strengthening the Structure	Tightening to a not determinate degree.	P.H. Jackson San Francisco	1888	U.S. Pat. No. 375,999 (1888)	yes	no	Many methods for stretching the reinforcement.
2	Reduction of the concrete tensile stress to a given limit and reduction of cracking	Counteraction due to stretching the tensile reinforcement, being not or only partly effective since initial prestress too small and losses not or only partly considered.	a. J. Mandl Vienna	1896	Journal Aust, Ass. Eng. & Arch (Z.D.Oe.I&A.V.)	yes	no	Stretching before concreting.
			b. M. Koenen Berlin	1907	Central-Journal Germ. Bldg. Ass. (Zentral Bl.d. D. Bau Verw.)			Stretching by hydraulic jacks before concreting.
			c. J.G.E. Lund Bjorn	1907	U.S. Pat. No. 1,020,578 (1912)	no	yes	Rods having threads tightened by nuts between prefab blocks.
			d. C.R. Steiner Gridley-Cel., Cal.	1908	U.S. Pat. No. 903,909 (1908)			Rods having threads tightened by nuts against green concrete and afterwards stretched again.
3	Guaranteed Cracklessness	Counteraction by full prestressing, the stretching force being of such magnitude that no tensile stress occurs when under working load, considering the greatest possible loss of prestress.	a. Rich, H. Dill Alexandria, Nebr.	1923 1925	U.S. Pat. No. 1,684,663 (1928)	no	yes	Destruction of bond by coating.
			b. W. H. Hewett Minneapolis, MN	1927	U.S. Pat. No. 1,818,254 (1931)			Similar to Dill's proposition.
			c. E. Freyssinet Neuilly-sur Seine, France	1928	U.S. Pat. No. 2,080,074 (1937)	yes	no	High strength steel or wire stretched, reinforcement substantially reduced.
			d. Thom. E. Nichols Hornell, N.Y.	1931	U.S. Pat. No. 2,035,977 (1936)			Tensile reinforcement in excess of usual requirements.
			e. F.O. Anderegg, Newark, Ohio	1934	U.S. Pat. No. 2,075,633 (1937)	no	yes	Tensioning tie rods extending through perforated ceramic blocks.
4	Extended applicability (Increase of span).	Tensioned ties in combination with normal reinforced concrete.	a. F. Dischinger Berlin	1934	Brit. Pat. No. 464,361 (1937)	no	yes	Ties, hanging in curved lines, engage externally the reinforced concrete elements.
			b. U. Finsterwalder Berlin	1936	U.S. Pat. No. 2,155,121 (1941)			
5	Reduction of cracking (similar to 2 but effective.)	Partial counteraction by combination of an effectively stretched and an unstretched tensile reinforcement. Partial prestressing.	F. Emperger Vienna	1939	U.S. Pat. No. 2,255,022 (1941)	yes	no	Unstretched main reinforcement in usual manner and bonded additional prestressed rods of superior strength.
6	Saving Steel		P.W. Abeles London	1940	Brit. Pat. No. 541,835 (1941)	yes	yes	The tensile reinforcement substantially reduced)thus differing form 5) by the use of high strength steel or wire also for the unstretched reinforcement.
		Partial counteraction by partial prestressing high strength steel.		1942	Brit. Pat. No. 554,693 (1943)			As before but the whole reinforcement prestressed to the same or to different extent.

Table A.1 The Various Systems of Prestressing for Structures. (Abeles 1959)

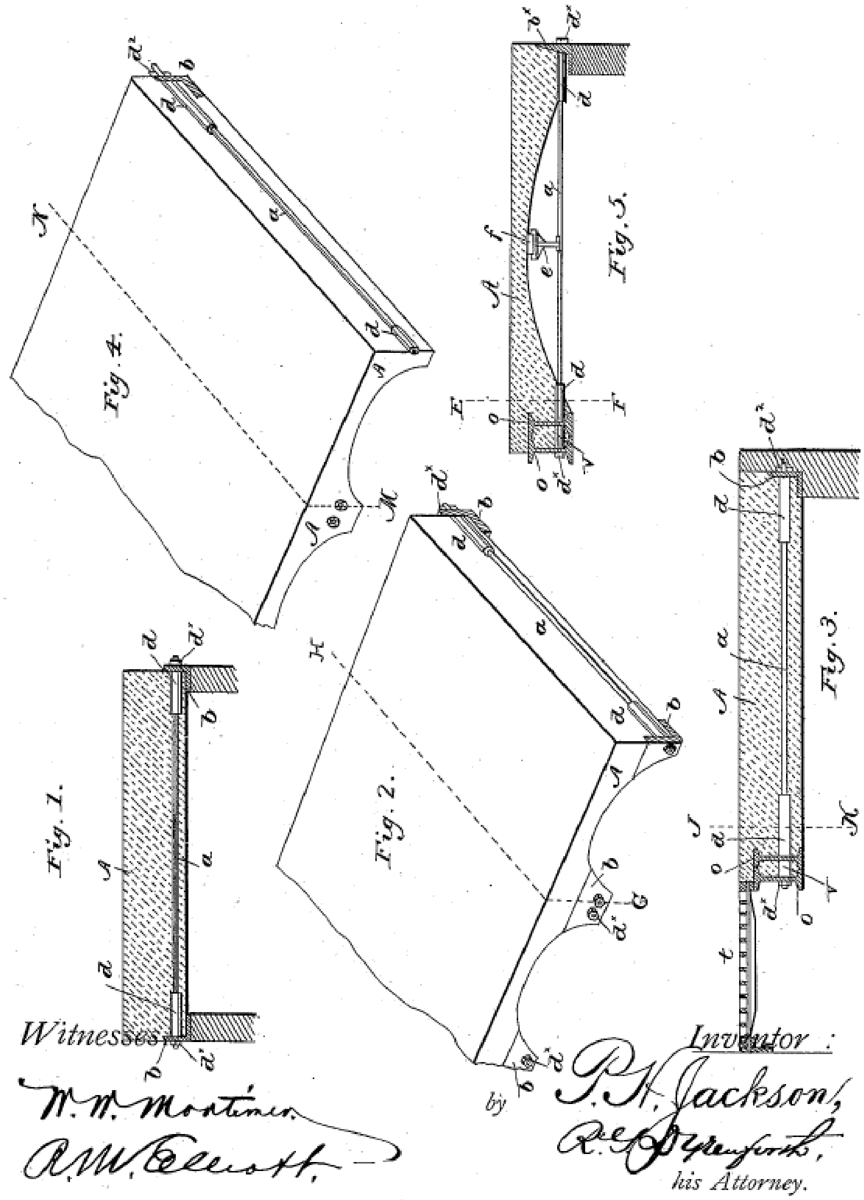
A.1.1 US 375,999

(No Model.) 2 Sheets—Sheet 1.

P. H. JACKSON.

CONSTRUCTION OF ARTIFICIAL STONE OR CONCRETE PAVEMENTS.

No. 375,999. Patented Jan. 3, 1888.



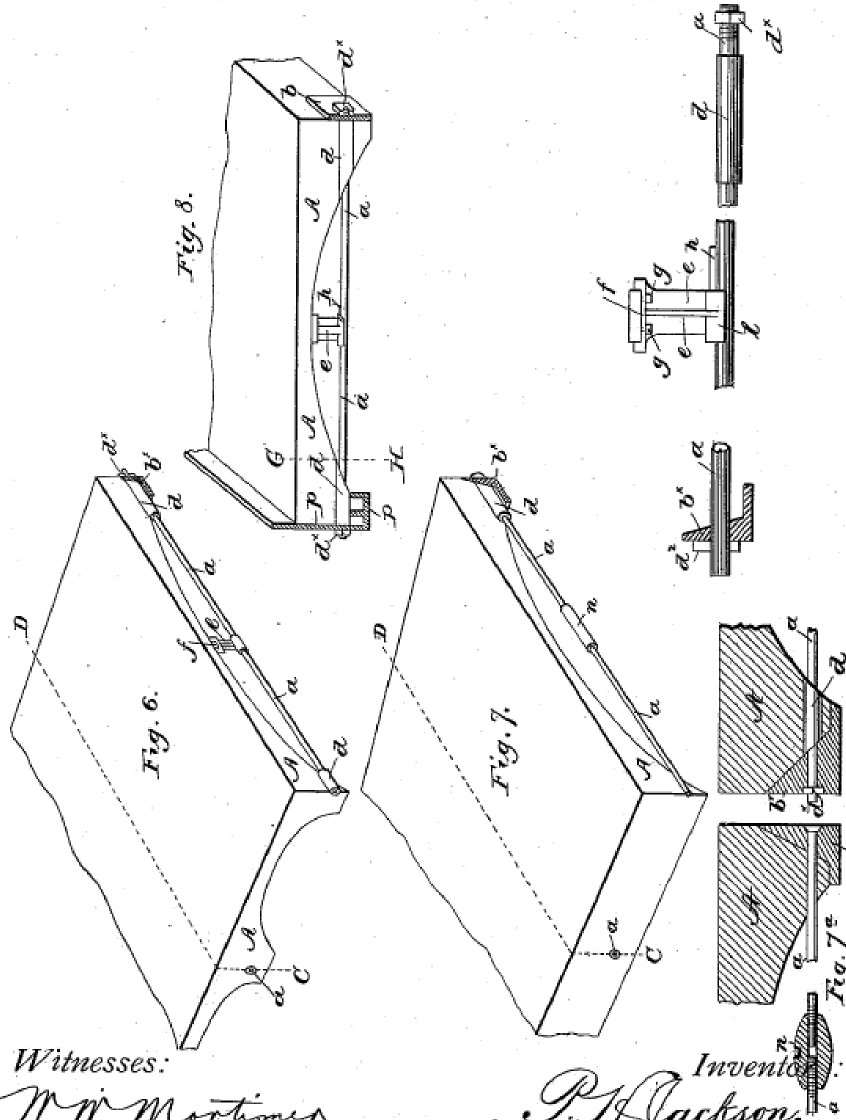
N. PETERS, Photo-Lithographer, Washington, D. C.

(No Model.) 2 Sheets—Sheet 2.

P. H. JACKSON.

CONSTRUCTION OF ARTIFICIAL STONE OR CONCRETE PAVEMENTS.

No. 375,999. Patented Jan. 3, 1888.



Witnesses:
W. W. Mortimer
R. M. Elliott

Inventor:
by P. H. Jackson,
R. F. Dyrenforth
his Attorney.

UNITED STATES PATENT OFFICE.

PETER H. JACKSON, OF SAN FRANCISCO, CALIFORNIA.

CONSTRUCTION OF ARTIFICIAL-STONE OR CONCRETE PAVEMENTS.

SPECIFICATION forming part of Letters Patent No. 375,999, dated January 3, 1888.

Application filed October 27, 1886. Serial No. 217,372. (No model.)

To all whom it may concern:

Be it known that I, PETER H. JACKSON, of San Francisco, State of California, have invented certain new and useful Improvements in the Construction of Artificial-Stone or Concrete Sidewalks, Floors, Roofs, &c.; and I declare the following to be a full, clear, and exact description thereof, sufficient to enable any person skilled in the art to which my invention belongs to make and use the same, reference being had to the accompanying drawings, forming part of this specification.

The invention relates to artificial-stone or concrete sidewalks, floors, or roofs.

The object of the invention is to strengthen and render more durable sidewalks, floors, roofs, and similar bodies constructed of artificial stone, concrete, or like material.

The invention consists in the combination of a sidewalk, floor, roof, or similar body constructed of artificial stone, concrete, or like material, with a series of arches, the footings of the arches being connected by ties provided with sleeves and skewbacks at their ends, and means whereby the skewbacks are forced against the material, the span of the arch decreased, and the structure strengthened; furthermore, in a sidewalk, floor, roof, or like body constructed of artificial stone, concrete, or like material, with a series of arches, the footings of the arches being connected by ties provided with sleeves and skewbacks at their ends, and intermediate of their ends with adjusting devices; furthermore, of a sidewalk, floor, roof, or like body constructed of artificial stone, concrete, or like material, with a series of arches, and having embedded in the footings of the arches ties provided with sleeves and skewbacks at their ends, the ends of the ties being screw-threaded and provided with keys, whereby the skewbacks may be thrust against the material; and, finally, in various novel details of construction whereby the effectiveness of the structure is insured and the object of the invention is attained.

To resist the tensile strain, one or more metallic ties are built in at or near the bottom of the arch and along its length, with their ends projecting through. The ties are united to the inclosing material by hydraulic or other strong cement, or are held by the plastic material of which the arch is composed. That part of the tie which projects through the ends of the arch is provided with screw-threads and nuts or keys, wedges, or similar well-known means, and pass through a plate or skewback at each end, the end of the arch fitting against the plate or skewback, and when the nut, key, or wedge is tightened on the outside of the abutting plate or skewback it presses the plate against the end of the arch, making it an abutment to resist the horizontal thrust; also, to prevent the material surrounding the tie along its length from slipping or sliding over it. By this arrangement for ordinary use plain bar, rod, plate, or band iron, or other shaped metal made at rolling-mills may be used as tie metal at or near the bottom of the arch without the usual expensive preparation of corrugating, roughening, indenting, or forming raised portions or irregular surfaces on the metal, or by cross-stops or by pins or any other preparation of the tie to make it hold over its length to the inclosing material when cemented to it.

Plain bar, rod, plate, or band iron or steel ties, without the preparation described for holding, and without the arrangement of the end abutting plates or skewbacks, but only cemented along the bottom of the arch, will not hold from sliding through the inclosing material when subjected to severe tensile strain.

A metallic tie at the bottom of an arch and extending along its length to be subjected to transverse strain must of necessity be cemented to the inclosing material practically over its length and more securely held at the center of its length, which is the place subjected to the greatest transverse strain, to prevent its breaking away in its connection to the top when the arch is undergoing deflection.

With metallic ties that have been prepared over their length to hold to the inclosing material of the arch at the bottom, as described, and with the assistance of the skewbacks or abutting plates and the means for forcing them against the ends of the arch, the holding of the tie to the inclosing material is largely increased and is adapted to places of severe trial, as that of arches of slight rise and long span.

In front of buildings area-spaces have to be provided between them and the sidewalk. In floors openings are left for stairways, and for pavements and platforms over excavations the ends of the arches have to be supported, and for this purpose girders are used either at one or both ends. Where one end of the arch is supported by a girder and the other end by a wall or other support, the end without the girder-support requires skewbacks, &c., as described, the other end of the arches abutting against the side of the girder, the girder resisting the thrust of the arch and supporting their ends, the side of the girder taking the place of the abutting plates or skewbacks, the ends of the ties passing through the girder, with the nuts, keys, or wedges tightened on the outside, thus firmly holding the girder to the arches, and should the girder be of two or more metal beams it firmly holds them together, being held in place by the ties, which pass through them, and by filling in the space between the beams with artificial stone, concrete, or other plastic material, when hardened, any number of beams forming the girder become as one in support of the load and prevent leakage from the top between the beams forming the girder.

When the metallic ties are cemented to the inclosing material along their length, they are incorporated with the mass as long as they hold to it, and any pulling force applied at the end of the tie and compressively resisted by the abutting end of the arch, which is the fulcrum, is not tensilely felt by the tie in the inclosing material while held to it. Should the tie not be straight or bent, any amount of pulling force on the end of the tie would fail to change it as long as the inclosing material held it in its bond. In order to produce tensile strain on the tie at a suitable distance from the ends, I use tubes or sleeves that will fit closely on the tie, one being slipped over each end of the tie with the outer end against the abutting plate, skewback, or side of girder, the tie passing through them, the inclosing material at the end of the arch being cemented to the sleeves and the tie being free to slide on the inside. After the concrete or other material of the arch has become hard and strong and the tie tensilely strained inside of the sleeves, that part of the bottom of the arch between the inside ends of the sleeves is compressed and the arch strengthened. These sleeves or coverings of the tie may be of any shape to conform to the shape of the tie, and are preferred to be of metal, or may be of any other material that will separate the tie from the material of the arch and permit it to slip through it, such as thick cloth, paper, black-lead, clay, &c.

Figures 1 to 4, inclusive, represent arches extending in a cross direction, and in the bottoms between and at their ends along their length are metallic ties embedded and cemented to the inclosing material with skewbacks, sleeves, &c.

Fig. 1 is a longitudinal section of an arch, A, on the line G H, Fig. 2.

a is a metallic tie built in the material at the bottom between the arches and along the length, extending through the sleeves *d d* at the ends, and through abutting plates *b b*, and having screw-threads on its ends, with the nuts $d^{\times}$ $d^{\times}$, when screwed up, pressed against the abutting plates and ends of the arch.

Fig. 2 is a perspective view, of which Fig. 1 is the longitudinal section, showing the ends of the arches A with the ties, sleeves, and abutting plates as described for Fig. 1.

Fig. 3 is a longitudinal section of an arch from J K, being on the line M N of Fig. 4. At one end is shown a girder composed of two metallic beams, *o o*, which may be of one, two, three, or more beams, as required for strength, or may be of any other form or kind of girder. This girder supports the ends of the arches A, which abut against it, the metallic tie *a* passing through the webs of both beams *o o*, and when the nut on the outside is screwed up after the arches are formed and become hard and strong it presses the girder against the end of the arches, making an abutment and taking the place of abutting plates or skewbacks. *d d* are the sleeves or coverings of the ties. Upon the other end of the arch is an abutting plate, *b*, with the wedge or tapering key d^2 forced into the slot in the tie and pressing against the abutting plate *b*, serving the same purpose as the nut on the other end of the tie. A section of illuminating-tile, *t*, is shown extending out from the front of the building, upon which one end rests, to the sidewalk and over on the girder covering the area-space, the space V between the beams *o o* being filled in with concrete or like plastic material.

Fig. 4 shows a view cut off on the line J K of Fig. 3, in order that the ends of the arches may be seen.

In Figs. 5, 6, 7, and 8 the arches with ties extend the long way instead of crosswise, as in the preceding figures. The metallic ties *a* pass through the sleeves at the ends of the arches, and by screwing up the nuts or forcing the keys against the abutting plates, skewbacks, or girder the arch is cambered and the bottom or intrados of the arch is compressed, increasing the strength of the arch, which could not be done if the sleeves or coverings of the ties were omitted.

Fig. 5 is a longitudinal section of the arch A on the line C D of either Fig. 6 or 7, shown with the girder composed of two metallic beams, *o o*, the tie *a* passing through them, and the space V between the beams filled with artificial stone, concrete, or other plastic material, when hardened uniting the beams as one, and by screwing up the nuts $d^{\times}$ $d^{\times}$ it shortens the tie between the nuts and cambers the arch. $b^{\times}$ is a skewback. In many cases access cannot be had to the end nuts or keys to tighten and strain the ties by reason of the ends being covered by masonry or brick-work, and to overcome this over the strut *e*, between it and the under side of the arch, is shown the tapering key or wedge *f*, which, when forced in, increases the distance between the tie and the under side of the arch, which cambers the arch and strains the tie, thereby increasing the strength of the arch to sustain a load on its top; or, in place of the strut key or wedge, a turn-buckle is used to shorten the tie.

Fig. 6 is a perspective view on the line E F, Fig. 5, showing the sides and ends to be direct and transverse arches, as described in my Patent No. 339,296, but with the improvement for cambering and strengthening the arches by means of the sleeves *d d* on the ties *a*, as well as the key *f* over the strut *e*.

Fig. 7 is a perspective view of a direct arch, or arched only in one direction over its length from the line E F, Fig. 5, and extending to the skewback $b^{\times}$. *n* is a turn-buckle on the tie to shorten it.

Fig. 7^a is a longitudinal section of turn-buckle *n* and skewback.

Fig. 7^b is a longitudinal section of a recessed skewback, $b^{\times}$, holding an immovable nut, $d^{\times}$. The nuts on opposite ends act on reverse threads. The distance between nuts is shortened by turning the tie *a* in the middle of its length by tongs, thereby cambering the arch, producing compression at the intrados.

Fig. 8 is a longitudinal section representing the same construction as Figs. 6 and 7 on the line C D, and having a girder, *p*, of L shape on one end and an abutting plate, *b*, at the other. The strut *e* has a tapering key, *h*, at the bottom between it and the tie.

Fig. 9 is an enlarged view of the parts. The tie *a* is shown with the nut $d^{\times}$ on one end, and at the other end a tapering key, d', driven through the tie. The strut *e* shows some of the different methods for increasing the distance between the tie and the crown of the arch. One is shown with the tapering key *f* at the top, and another is shown to be with screws *g g*, passing through the top of the strut and pressing against a plate, which forces up the arch. Another is shown with the key *h* between the tie and the under part of the strut.

The strut may be made in two pieces in height having a tapering screw, key, or wedge between, by which the distance between the tie and arch may be increased, as in the other methods.

Having thus described my invention, what I claim as new, and desire to secure by Letters Patent, is—

1. The combination of a sidewalk, floor, roof, or similar body constructed of artificial stone, concrete, or like material, with a series of arches, the footings of the arches being connected by ties provided with sleeves and skewbacks at their ends, and means, substantially as described, whereby the skewbacks are forced against the material, the span of the arch decreased, and the structure strengthened.

2. A sidewalk, floor, roof, or similar body constructed of artificial stone, concrete, or like material, with a series of arches, the footings of the arches being connected by ties provided with sleeves and skewbacks at their ends, and intermediate of their ends with adjusting devices, substantially as described.

3. A sidewalk, floor, roof, or like body of artificial stone, concrete, or like material, with a series of arches, and having embedded in the footings of the arches ties provided with sleeves and skewbacks at their ends, the ends of the ties being screw-threaded and provided with keys, whereby the skewbacks may be thrust against the material, substantially as described.

4. A sidewalk, floor, or roof constructed with artificial stone or concrete, arches with longitudinal ties to resist the tensile strain, with the ends extending through sleeves or coverings, by which that part of the ties may slide independent of the material which it passes through, in combination with screws, nuts, keys, wedges, turn-buckles, or the like, by which tension on the tie may be increased and the material acted upon compressed, substantially as herein described.

PETER H. JACKSON.

Witnesses:
JAMES B. LANE,
WM. MAYER.

Corrections in Letters Patent No. 375,999.

It is hereby certified that in Letters Patent No. 375,999, granted January 3, 1888, upon the application of Peter H. Jackson, of San Francisco, California, for an improvement in "The Construction of Artificial Stone or Concrete Pavement," errors appear in the printed specification requiring correction as follows: In line 40, page 1, the word "and" should have been printed *or;* and on page 3, line 75, the words *having nuts, or being* should have been inserted before the word "provided;" and that the said Letters Patent should be read with these corrections therein that the same may conform to the record of the case in the Patent Office.

Signed, countersigned, and sealed this 14th day of February, A. D. 1888.

[SEAL.]

D. L. HAWKINS.
Acting Secretary of the Interior.

Countersigned:
BENTON J. HALL,
Commissioner of Patents.

A.1.2 US 2,080,074

May 11, 1937. E. FREYSSINET ET AL 2,080,074

PIECE OF REENFORCED CONCRETE

Filed March 8, 1934

Fig. 1.

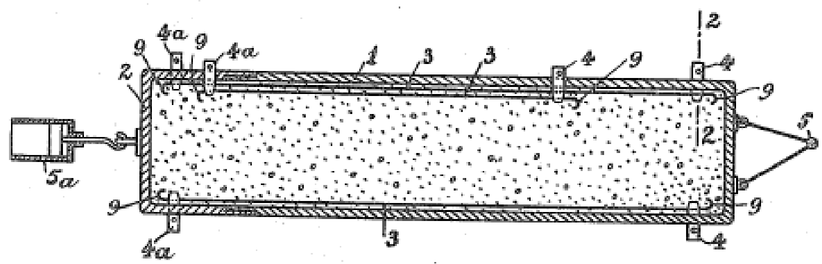


Fig. 2.

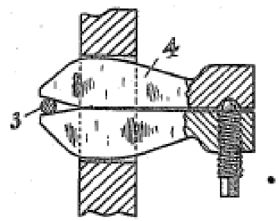


Fig. 3.

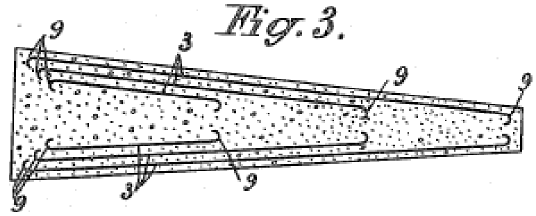
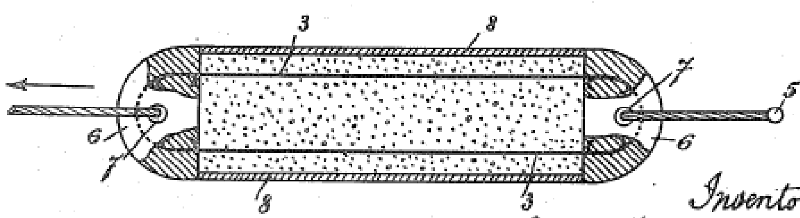


Fig. 4.



Inventors:
Eugène Freyssinet and Jean Séailles
by Mauro & Lewis,
attorneys

Patented May 11, 1937 **2,080,074**

UNITED STATES PATENT OFFICE

2,080,074

PIECE OF REENFORCED CONCRETE

Eugène Freyssinet, Neuilly-sur-Seine, and Jean Séailles, Paris, France

Application March 8, 1934, Serial No. 714,724
In France October 2, 1928

7 Claims. (Cl. 72—61)

The present application is a continuation in part of the U. S. patent application Ser. No. 395,297, filed Sept. 26, 1929 in which we described a process of improving the manufacture of reenforced concrete articles by subjecting to a preliminary tension the metallic reenforcements so as to obtain compressive stresses in the concrete of these articles. The present invention concerns the permanently improved articles obtained by means of this process.

It has been proposed to subject the reenforcements of reenforced concrete articles to preliminary tensions before pouring the concrete into the moulds and then to transfer this tension of the reenforcements to the concrete after setting and hardening of the latter, by releasing the devices by means of which the reenforcements were subjected to tension. However, if special precautions are not taken, no permanent improvement of reenforced concrete articles can be obtained in this way. This is due to the fact that, when the tensions of the reenforcements are transferred to the concrete, numerous causes intervene for reducing this tension:

First of all, in the course of the setting and hardening of the concrete, the dimensions of the latter decrease, due to a certain phenomenon, which is called "shrinkage".

Secondly, concrete compressed by reenforcements that have been subjected to tension is elastically deformed.

Furthermore, a certain sliding displacement of the reenforcements with respect to concrete may occur in spite of anchoring members disposed on the reenforcements, until, due to the pressing together of the particles of concrete by the anchoring members, in the vicinity of these members, this sliding displacement ceases.

Finally, we discovered that, in concretes subjected to constant stresses, the concrete mass undergoes a progressive reduction of length analogous to a kind of shrinkage proportional to the stresses and which may be numerically more important than ordinary shrinkage, in the case of high tensions (see report of the session of September 22, 1926 of the "Commission Technique de la Chambre Syndicale des Constructeurs en Ciment Armé de France; Report of the Reenforced Concrete Convention, Liéges 1930; Report of the Convention of the Association Internationale des Ponts et Charpentes, Paris, 1932).

The sum of these drops of tension can amount to 30 kilogrammes per square millimetre, in the case of very resistant concretes, and even much more, in the case of ordinary concretes. Consequently, if the tensions to which the reenforcements are subjected are lower than 30 or 40 kilogrammes per square millimetre, a time will come when these tensions will be reduced to zero by the total shrinkage of the concrete, so that the increase of resistance which was intended to be obtained by subjecting the reenforcements to tension is destroyed and constructions including such concrete pieces may be ruined.

Now, the reenforcements that are employed at the present time are made of metals the elastic limit of which is, as a rule, much lower than 40 kilogrammes per square millimetre. In ordinary reenforced concrete, it is not known how to utilize the resistance of steels having higher elastic limits and the use of such steels is therefore unnecessary. The present regulations concerning concrete advise not to exceed stresses of 16,000 lbs. per sq. in. (which corresponds to 11.20 kilogrammes per square millimetre) for all kinds of steels (see Hool and Johnson "Concrete Engineer's Handbook", page 37, section 56). Therefore, the use of steels of high elastic limit has not developed in the ordinary technic of reenforced concrete, the fact being that the reenforcements utilized in the ordinary technic are of relatively low elastic limit. Now, with such reenforcements, it is impossible to bring into play tensile stresses sufficiently high for obtaining permanent improvements.

The object of the present invention is to provide reenforced concrete objects that are improved in a permanent manner, owing to the utilization of reenforcements having a sufficiently high elastic limit in order that these reenforcements may be subjected to preliminary tensions which considerably exceed the sum of the possible drops of tension and that, account being taken of these drops of tension, there may remain in these reenforcements high permanent tensions which are an important portion of the initial tension imparted to the reenforcements.

Therefore, in a reenforced concrete article according to the present invention, the concrete of which has set and hardened and has undergone all the shrinkages and reductions of length above referred to, there remain permanent tensions of the reenforcements, which exert on the concrete of the article permanent compressions. Owing to a suitable distribution of the tensioned reenforcements in the concrete, these compressive stresses are opposed to the stresses created in the concrete article by the external forces acting thereon, so as to compensate them.

The reenforcements utilized according to the present invention are made of metals having elastic limits which are considerably greater than 40 kilogs. per square millimetre, so that they may be subjected to tensions at least equal to this value and generally much higher, say 60 kilogs., 80 kilogs. and even more. Of course the elastic limit can be increased by suitably treating the metal, for instance by previously drawing or tempering it. The increase of the elastic limit may also result from the preliminary tensioning treatment itself, provided, of course, that the preliminary tension is lower than the ultimate strength of the metal. The reenforcements that are in contact with the concrete may be provided with anchoring members for preventing them from sliding with respect to the concrete. However, in most cases, the adhesion of reinforcement bars to concrete constitutes a sufficient anchoring.

Figs. 1 to 4 of the accompanying drawing show, by way of example, two embodiments of devices serving to subject the reenforcements to tension.

In Fig. 1, the mould 1, in which concrete is to be poured, comprises, at one end, a movable part 2. The ends of the reenforcements 3, which may be of unequal lengths and which are distributed in the concrete mass according to the diagram of the stresses to be created, are fixed, on the one hand to the mould through any suitable means such as gripping members 4 holding tightly the ends of the reenforcements (see the detail view of Fig. 2 which is a section on the line 2—2 of Fig. 1) or any other equivalent organ, and, on the other hand to the movable part 2 by means of similar gripping members 4*a*. This movable part 2 is connected to the piston of a powerful hydraulic jack 5*a* the cylinder of which is stationary while the mould is fixed, at the opposite end, to a stationary point 5. The reenforcements, made of a metal of high elastic limit, are first subjected to the required preliminary tensions by admitting into the jack a liquid under suitable pressure, which displaces the movable part 2 and stretches the reenforcements. Concrete is then poured into the mould, the reenforcements being kept tensioned. When concrete has set and hardened, the pressure in jack 5*a* is brought back to zero and consequently the tension of reenforcements 3 is transferred to the concrete. The movable part 2 is then removed, after the means 4 for fixing the reenforcements have been removed and the concrete piece can be taken off from the mould. Reenforcements of unequal lengths must be used when the cross section of the piece is not uniform in order that concrete may nevertheless be subjected to a uniform compression. There is for instance the case of a post of the shape of a frustum of a cone or of a frustum of a pyramid. This post will be provided with tensioned reenforcements 3 (Fig. 3) of unequal lengths. The number of these reenforcements in a cross section of the post is larger at the big end of said post than at the small end thereof. With tensioned reenforcements all of the same length, it would be impossible to subject the concrete to high compressive stresses without risk of crushing the concrete at the small end of the post.

In the example of Fig. 4, the ends of the reenforcements 3 are imbedded in blocks of concrete 6 by providing in said blocks suitable gripping means or means through which the blocks can be pulled away from each other, for instance holes such as 7. Once the concrete of the blocks has set and hardened and is sufficiently strong, the reenforcements 3 are subjected to the required tensions by moving blocks 6 away from each other, for instance by means of powerful hydraulic jacks. The reinforcements being thus kept in the tensioned state, we dispose between blocks 6 walls 8 forming the mould and concrete is poured thereinto. When concrete has set and hardened, walls 8 are removed and the pressure exerted by the hydraulic jacks is released. The reenforcements tend to shrink, and thus to bring blocks 6 toward each other, thus exerting a compression on the concrete in the mould.

In the example of Figs. 1, 2, and 3, anchoring members prevent the tensioned reenforcements from sliding with respect to the concrete. These members will consist of hooks 9 provided at the ends of the reenforcements or of any other organ, either integral with or fixed to said reenforcements, for instance local thickened portions thereof.

In the example of Fig. 4, the anchoring members consist of the two concrete blocks 6, in which the reenforcements will be, in turn anchored, by means of hooks or the equivalent.

For practical purposes, we generally use carbon steels in the form of wires of diameters ranging between 6 and 15 millimetres, cooled rather suddenly in the course of rolling, so that the ultimate strength of these steels ranges between 90 and 100 kilogs. per square millimetre and their elastic limit ranges between 45 and 50 kilogrammes per square millimetre. This elastic limit is increased up to 80–90 kilogrammes per square millimetre. We may also use steels that have been subjected to thermic treatments so that their ultimate strengths may be as high as 160 kgs. per square millimetre, particularly in the case of special steels, such as silicon steels. The tensions that give the best results range between two-thirds and three-fourths of the elastic limit of the metal, so that after all the drops of tension above referred to, there remain permanent tensile stresses of at least 30 or 40 kilogrammes per square millimeter. In such products, the normal working fatigue should range between ⅓ and 2/5 of the ultimate strength of the steels, which permits of subjecting the steels to stresses of 50 and 60 kilogrammes per square millimetre and even more without any risk of cracking of the concrete.

The products obtained according to the present invention differ from ordinary reenforced concrete products in that their strength is considerably higher than that of an ordinary reenforced concrete product of the same weight and price or their weight and cost are much lower than those of an ordinary reenforced concrete product of the same strength. More especially, the products according to the present invention have an extraordinary resistance to shearing and twisting stresses, often five times greater than those of ordinary reenforced concrete elements of the same section. Their resistance to alternating stresses is practically unlimited, while ordinary reenforced concrete has but a rather bad resistance to this kind of stresses.

What we claim is:

1. A piece of reenforced concrete which comprises set and hardened concrete having undergone shrinkage, and reenforcements of a steel having a high elastic limit in permanent tension anchored in said concrete and adhering thereto along their whole length, whereby said concrete is permanently subjected to compressive stresses.

2. A piece of reenforced concrete which comprises set and hardened concrete having undergone shrinkage, and reenforcements of a steel having a high elastic limit in permanent tension anchored in said concrete and adhering thereto along their whole length, said reenforcements being tensioned and distributed in accordance with the diagram of the tensile stresses that are to be developed in the piece by determined external forces acting thereon, so as to produce in the concrete permanent compressive stresses that balance these tensile stresses.

3. A piece of reenforced concrete which comprises permanently compressed set and hardened concrete having undergone shrinkage, and reenforcements of a metal the elastic limit of which is higher than 40 kilogrammes per square millimeter, anchored in said concrete and adhering thereto along their whole length, said reenforcements being in permanent tension.

4. A piece of reenforced concrete which comprises permanently compressed set and hardened concrete having undergone shrinkage, and reenforcements of a metal the elastic limit of which is higher than 40 kilogrammes per square millimeter, anchored in said concrete and adhering thereto along their whole length, said reenforcements being in permanent tension at a rate of at least 10 kilogrammes per square millimeter.

5. A piece of reenforced concrete which comprises permanently compressed set and hardened concrete having undergone shrinkage, and reenforcements of a metal the elastic limit of which is higher than 40 kilogrammes per suare millimeter, anchored in said concrete and adhering thereto along their whole length, said reenforcements being permanently tensioned, at a rate of at least 10 kilogrammes per square millimeter, and distributed in accordance with the diagram of the tensile stresses that are to be developed in the piece by determined external forces acting thereon, so as to produce in the concrete permanent compressive stresses that balance these tensile stresses.

6. A piece of reenforced concrete which comprises permanently compressed set and hardened concrete having undergone shrinkage, and a plurality of rectilinear reenforcements made of a metal the elastic limit of which is higher than 40 kilogrammes per square millimeter, anchored in said concrete and adhering therewith along their whole length, said reenforcements being in permanent tension at a rate of at least 10 kilogrammes per square millimeter, the length, distribution and tension of these reenforcements being so chosen, in accordance with the system of tensile stresses to be developed in the piece by determined external forces acting thereon, as to produce in the concrete permanent compressive stresses that balance these tensile stresses.

7. A piece of reenforced concrete which comprises permanently compressed set and hardened concrete having undergone shrinkage, and a plurality of parallel rectilinear reenforcements made of a metal the elastic limit of which is higher than 40 kilogrammes per square millimeter, anchored in said concrete and adhering therewith along their whole length, said reenforcements being in permanent tension at a rate of at least 10 kilogrammes per square millimeter, the respective lengths and tensions of these reinforcements being so chosen, in accordance with the system of tensile stresses that are to be developed in the piece by determined external forces acting thereon, as to produce in the concrete permanent compressive stresses that balance these tensile stresses.

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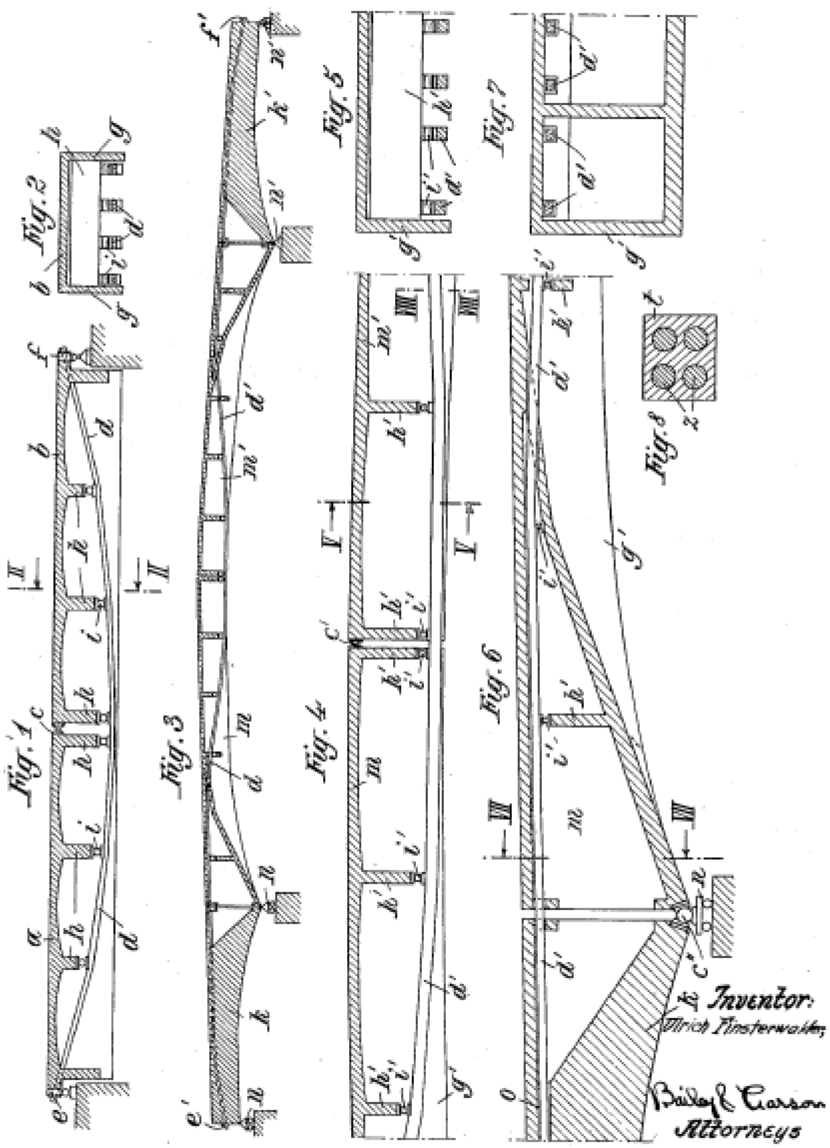
JEAN SÉAILLES.

A.1.3 US 2,155,121

April 18, 1939. U. FINSTERWALDER 2,155,121

FERRO-CONCRETE BEAM

Filed June 10, 1937



Patented Apr. 18, 1939

2,155,121

UNITED STATES PATENT OFFICE

2,155,121

FERRO-CONCRETE BEAM

Ulrich Finsterwalder, Berlin-Wilmersdorf, Germany

Application June 10, 1937, Serial No. 147,579
In Germany January 11, 1936

10 Claims. (Cl. 72—56)

The use of ferro-concrete beams for structures of wide-span, especially for bridges, is limited as regards width of span. The structure is generally manufactured as a plate beam with depending longitudinal webs spaced about 3 metres apart. For manufacturing reasons the webs have a minimum breadth of about 25 centimetres, while the thickness of the plate is determined by the loading.

With increasing width of span of the beam and constant relation of the depth of the structure to the span-width the bending stress increases and thus the quantity of iron necessary with uniform weight per metre of the structure increases linearly. Even with very small span widths of about 13 metres the web-breadth of 25 centimetres no longer suffices for the disposal of the necessary tension irons. Therefore the webs must be broadened, whereby the weight of the structure, calculated on the area of surface, is substantially increased, so that with a freely supported beam the limit is reached with a span width of about 25 metres.

Entirely aside therefrom, in known constructions, even with span widths below those given, the consumption of material is very great, resulting in high cost of production and low efficiency.

According to the invention, which eliminates the aforesaid drawbacks, the beam is formed of jointed concrete sections which have none of the usual reinforcing rods passing therethrough and are connected by an external tension bar which replaces the internal reinforcing rods. The beam is subdivided preferably at the mid-point, and comprises two similar jointed sections.

It is particularly advantageous to support the tension bar on roller bearings arranged at the lower ends of integral cross-ribs which depend from the concrete sections so that the stresses in the tension bar and in the concrete are substantially independent of alterations of form of these structural members.

Hereby without any pre-stressing of the tension bar and irrespective of alteration of form of the concrete there is obtained a constantly uniform tensile condition in the bar and in the beam. By suitable choice of the level of the tension bar the tensile condition can be controlled at will. It is thus possible, for example, so to choose the level of the deflecting points for the tension bar, that balanced marginal stresses are set up in the ferro-concrete beam under the most unfavourable loading conditions.

With the invention further advantages are realised.

In contrast with other constructions, with the invention the depth of the structure available at the mid-point can be fully utilised. Above all, it is possible by the invention, with the concrete stresses permissible today, with the aid of freely supported bridge beams with $\frac{1}{15}$ structural depth, to span up to 70 metres, and with girder beams to span up to 120 metres. The structures provided can also be manufactured economically as their weight per square metre is only slightly greater than that of bridges of smaller width of span.

Two embodiments of the invention are illustrated in the accompanying drawing.

Fig. 1 is a longitudinal section of a freely supported ferro-concrete beam according to the invention.

Fig. 2 is a cross-section on the line I—I of Fig. 1.

Fig. 3 shows in longitudinal section a second embodiment of the invention in the form of a ferro-concrete bridge structure incorporating four concrete sections.

Fig. 4 shows also in longitudinal section but to a larger scale than Fig. 3 the middle section of the structure shown in Fig. 3.

Fig. 5 is a transverse section on the line V—V of Fig. 4.

Fig. 6 reproduces to a larger scale a fragment of the structure according to Fig. 3.

Fig. 7 is a transverse section on the line 7—7 of Fig. 6.

Fig. 8 shows in section a tension bar sheathed in ferro-concrete.

As appears from Fig. 1, the beam comprises two sections *a* and *b*, which are articulated to one another by means of a joint designated *c*.

The joint may be devised in different ways, following known teachings. In certain circumstances the simple severance of the beam and the interposition of lead interlays in the gap is sufficient.

The subdivision of the beam is effected, in this embodiment, at the mid-point. If required, it may be effected at another point.

For joining the beam sections there serves also a tension bar *d*, hereinafter particularly described, which is anchored at one end in the beam section *a* and at the other end in the beam section *b*.

The beam sections *a* and *b* comprise a continuous upper compression plate with depending longitudinal webs *g* and transverse ribs *h*. The compression plate is arched on its under face between successive transverse ribs *h*.

In the embodiment illustrated the tension bar *d* is led from the anchorage points over the lower ends of the several transverse ribs *h*, but the invention is not confined to the course of the tension bars represented in this example.

The transverse ribs *h* form deflecting points for the tension bar *d* and are provided at their lower ends with roller bearings *i* over which the tension bar is led. The roller bearings are equipped in usual manner with horizontal races and are also vertically adjustable.

The rollers need not be vertically adjustable at all the deflecting points. The arrangement may be such that only selected rollers are adjustable in vertical direction. In certain circumstances also vertical adjustment may be dispensed with, especially if the tension bar *d* is adapted for after-adjustment at the anchorage points *e*, *f*.

As will be apparent, the bending moment of the beam at the mid-point is taken up by the compression stress at the joint *c* and the tensile stress in the tension bar *d*.

In consequence of the rollers *i* being mounted for horizontal movement, the horizontal stress in the tension bar *d* is constant throughout. Thus, at any point of the beam, from the moment and from the known horizontal stress of the tension bar the leverage of the internal stresses can be determined, and by choice of the level of the deflecting points the level of the compression stress acting on the ferro-concrete beam can be freely chosen.

With unilateral loads the amount and the level of the compression stress are altered to a small extent. It is a matter of practical design, having regard to the circumstances of the particular case, so to fix the level of the deflecting points that with the most unfavourable loading conditions marginal stresses balanced as far as possible may be obtained. Depression due to plastic deformations of the beam does not occasion stresses, as the leverage of the tension bar in the beam and the leverage of the internal stresses are not altered. In case for any reason the beam is lifted too high or sags, it is immediately possible to effect a correction, for example by adjusting the level of the deflecting points. The tension bars *d* consist either of normal iron of suitable cross-section or of composite iron construction.

It is, however, desirable to form the tension bars of ferro-concrete, so that they may be well protected against the influences of the weather.

In the embodiments according to Figs. 1 to 7 it is assumed that the tension bars are formed of round iron sheathed with concrete.

In the example shown in section in Fig. 8, the tension bar consists of four round iron rods *z* enveloped by a sheath *t* of concrete of rectangular form and section.

The concrete sheath *t* is preferably formed after the erection of the bridge, in order to avoid the setting up of tension stresses in the concrete. Thus, before the sheathing of the bars a load may be applied to the bridge to the extent of the traffic load to be expected, and the tension bars sheathed with concrete under the conditions thus set up. This method offers the advantage that the concrete of the ferro-concrete tension bar is normally under compression, whereby elastic deformations of the tension bar are reduced.

In the embodiment according to Figs. 3 to 7 the ferro-concrete beam consists of four concrete sections, namely, two similar outer sections *k*, *k'* which are separated by gaps from the adjacent inner sections *m*, *m'*. The two inner sections *m*, *m'* are in turn separated from one another by a gap shown in Fig. 4 and connected by a joint *c'*. The formation of the junction of the inner and outer sections *k*, *m* and *k'*, *m'* is shown in Fig. 6. Also in this case a gap is interposed between the adjacent concrete sections and an articulated joint is provided. In this case, however, the joint is located on the underside of the sections. As appears from Fig. 3 three supports indicated at *n* are formed as roller bearings and that indicated at *n'* as a tilting bearing.

The round iron rods indicated at *d'* extend in this embodiment of the invention in a continuous course from the anchorage point *d'* on the left-hand outer section *k* to the anchorage point *f'* on the right-hand outer section *k'*. In both outer sections *k*, *k'* the tension bars are guided in suitable channels. In the region of the inner sections *m*, *m'* they run over rollers *i*, *i'* as in the embodiment of Fig. 1, the said rollers *i*, *i'* being disposed at the lower ends of the transverse ribs *h'*.

In other respects the construction of the inner sections *m*, *m'* is similar to that of the beam sections *a* and *b* in Fig. 1. Also the inner sections *m*, *m'* each comprise a continuous upper compression plate with depending longitudinal webs *g* and transverse ribs *h'*.

In the last described construction static determinativeness of the system is obtained, so that both the tensile stress in the tension bar and also the compression stress at the joint can be calculated without regard to alterations of form. The cross-sectional formation of the ferro-concrete beam altered as compared with the freely supported beam of Fig. 1 at the junction points according to Figs. 3 to 7 results from the stressing of the structure by negative moments.

In both embodiments the axes of the ferro-concrete beam and of the tension bar are at such an angle to one another that the actual transverse stress is substantially taken up. Thereby thrust stresses in the ferro-concrete beam are almost entirely avoided. Also tensile stresses in the concrete due to bending and to thrust are entirely or almost entirely avoided.

With regard to the form of the joint, the invention is not restricted to the arrangement of corporeal joints in the narrow sense. In certain circumstances, besides the forms described, also weakening of the cross-section at a determined point of the beam or crossed reinforcing irons or other means will suffice for forming in the beam a part which for the purposes of the invention will act practically like a joint.

I claim:—

1. A ferro-concrete beam comprising a compression plate formed by a pair of elongated members in end-to-end relationship, said members being adapted and arranged to be supported substantially at their outside ends, a plurality of transverse ribs attached beneath said members, said members and ribs being concrete, roller bearings on the lower ends of said transverse ribs, and an iron tension band engaged beneath said bearings, the ends of said bar being anchored in said members substantially at the outside ends thereof.

2. A ferro-concrete beam as claimed in claim 1, and extensible means between said roller bearings and the lower ends of said trans-

verse ribs, and means for adjusting the length of said extension means.

3. A ferro-concrete beam as claimed in claim 1, and means for adjusting the length of said bar.

4. A ferro-concrete beam as claimed in claim 1, and extensible means between said roller bearings and the lower ends of said transverse ribs, and means for adjusting the length of said extension means, and means for adjusting the length of said bar.

5. A ferro-concrete beam, comprising a compression plate formed by a pair of elongated concrete members hinged together at their inner ends each of said members having at least one downwardly extending portion, and at least one tension band, the ends of said band being secured to said members substantially at the outside ends thereof, respectively, said band being engaged beneath said downwardly extending portions.

6. A ferro-concrete beam, comprising a compression plate formed by a pair of elongated concrete members hinged together at their inner ends, each of said members having a plurality of downwardly extending portions, the portion nearest the inside end on each member being substantially longer than at least one of the other portions on each of said members, and at least one tension band, the ends of said band being secured to said members substantially at the outside ends thereof, respectively, said band being engaged beneath said portions.

7. A ferro-concrete beam as claimed in claim 5, and pivoted supports for said members at their outside ends.

8. A ferro-concrete beam, comprising concrete sections joined in end to end relation, and devoid of internal reinforcing rods passing through said sections, and at least one tension band disposing externally of and connecting said sections and extending lengthwise of the beam, said sections being formed with integral cross-ribs depending therefrom, roller bearings at the free ends of said cross-ribs, said tension band being engaged with said roller bearings.

9. A ferro-concrete beam as claimed in claim 5, the axis of said tension band extending at an inclination to the axes of the concrete members.

10. A ferro-concrete beam as claimed in claim 5, said members being arched on the underside.

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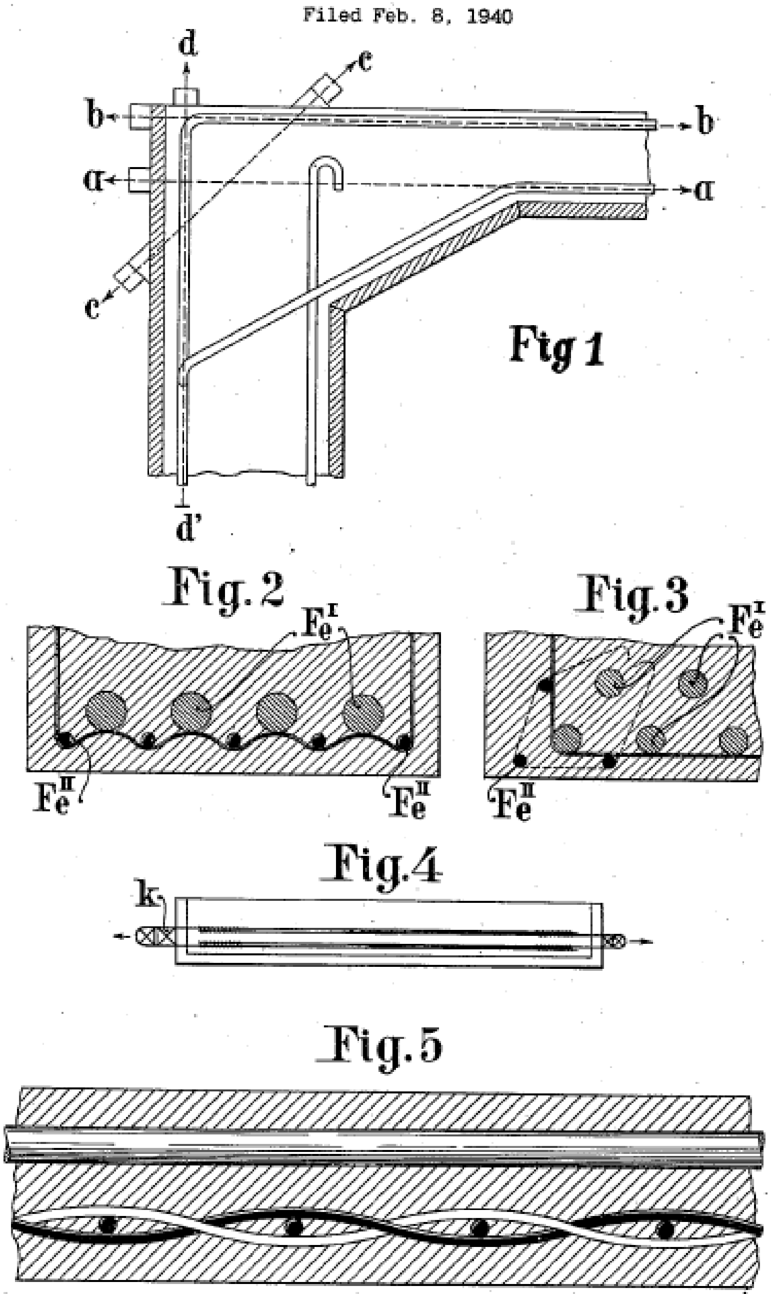
A.1.4 US 2,255,022

Sept. 2, 1941. F. EMPERGER 2,255,022

REINFORCED CONCRETE

Filed Feb. 8, 1940



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UNITED STATES PATENT OFFICE

2,255,022

REINFORCED CONCRETE

Fritz Emperger, Vienna, Germany, assignor of ten per cent to Joseph O. Ollier, New York, N. Y.

Application February 3, 1940, Serial No. 317,965
In Germany January 25, 1939

7 Claims. (Cl. 72—61)

The invention refers to objects or structures of reinforced concrete, such as girders, arches, frames, beams, and parts thereof.

The concrete of structures of such type is subject to cracks in zones where tensile stresses occur under load, and it has been suggested to prevent formation of such cracks and to increase the strength of the structure by preliminary or prestressing all the reinforcements which therefore had to be made of high quality steel. Considerable difficulties were met however, in manufacturing such concrete bodies or structures either in a plant or in situ, i. e. at the place where they were to be used, in that uniformly prestressing a great number of reinforcements requires complicated apparatus and great care in their use, and the great number of relatively thin reinforcements of high quality steel considerably increased the cost.

Taking a cylindrical reinforcement embedded in concrete, it has been shown (cf. O. Graf "Beton und Eisen," 1910, p. 177 and "Handbuch für Eisenbetonbau," 4th ed., first vol., p. 40) that the increase in plasticity of the concrete by reinforcement is very small, and the less the thicker the layer or body of concrete covering the reinforcement is. The maximum plastic or elastic deformation of a layer of 2 mm. thickness around the cylindrical reinforcement has been found to be 0.4% at most before rupture occurs, and if the thickness of the covering cylindrical layer or body amounted to 30 mm., the maximum elongation without rupture has been found to be only 0.2%. Taking a steel the elongation of which amounts to 0.2% at a stress of 400 kg./sq. cm., the covering layer of 30 mm. thickness will yield and form cracks if that stress of 400 kg./sq. cm. is exceeded.

Taking however a higher quality steel the elongation of which amounts to 1.4% at a stress of 2800 kg./sq. cm., experiments made by the inventor have shown that a relatively thin cover of concrete will be sufficiently plastic so as to crack only at the yield point of that high quality steel.

It has been suggested therefore to use high quality steel for reinforcements of concrete and to arrange them in such numbers and proximity to each other, furthermore to prestress them uniformly to such an extent that due to the bond of the set and shrunk concrete with the individual reinforcements, crack formation was prevented under predetermined maximum load. The compressive stresses exerted by the prestressed reinforcements through the bond upon the concrete were so high that they counterbalanced the tensile stresses exerted upon the concrete by the predetermined maximum load.

It has been found however that crack formation is not dangerous as long as cracks, when formed, are prevented from widening in continuous use and under recurrent load, and substantially close when the load is moved.

Hence it follows that the reinforcements should be prestressed only to such an extent that the compressive stresses exerted by them upon the concrete bonded with them suffices to substantially close the cracks under dead load.

Even if prestressing is limited in this way, or highly prestressed reinforcements are arranged in such distance from each other that the compressive stresses produced by them in the concrete bonded with them suffice to substantially close cracks produced by the live load, their number is still great and the difficulties as to uniformly prestressing them as well as their high cost persist.

It is therefore an object of the invention to reduce the cost of manufacturing in a plant or in situ of objects or structures of reinforced concrete the hair line cracking of which is reduced by prestressing those reinforcements.

It is another object of the invention to increase the strength and reduce the crack formation of objects or structures of reinforced concrete in which the reinforcements are prestressed.

It is still another object of the invention to secure more uniform prestressing of those reinforcements.

It is still another object of the invention to reduce the amount of high quality steel used in reinforced concrete the reinforcements of which are prestressed.

It is still a further object of the invention to prolong the life of objects or structures of reinforced concrete by reducing the width of cracks formed under predetermined maximum live load and causing the cracks to substantially close under dead load by the use of prestressed reinforcements.

It is still a further object of the invention to prevent hair line cracking in reinforced concrete structures.

It is still another object of the invention to improve the property, in particular the tensile strength of the concrete used in reinforced concrete structures.

According to the invention main reinforcements or rods are used and disposed in the usual manner, and in addition thereto prestressed other preferably rod-like reinforcements. While the main reinforcements preferably consist of ordinary steel as heretofore used for reinforcements, the additional reinforcements consist of a higher quality steel, i. e. a steel of superior tensile strength and yield point than the steel used for the main reinforcements. The number of additional reinforcements will be considerably smaller than that of higher quality steel reinforcements heretofore used in reinforced concrete objects in which all the reinforcements consisted of higher quality steel which had been prestressed.

Furthermore, uniform prestressing of the relatively small number of additional reinforcements can more easily be effected than prestressing of all the reinforcements of high quality steel. It is also possible to cut these additional reinforcements so as to cover only regions or zones in which tensile stresses under maximum load occur which might cause dangerously wide, or gaping cracks. In particular, the cut pieces of the additional reinforcements can be arranged in any desired manner and relative configuration so as to follow e. g. a curved or polygonal structure and its axis.

The objects and nature of the invention will be more clearly understood when the specification proceeds with reference to the drawing in which Fig. 1 shows more schematically in a section taken in the direction of the reinforcing bars or rods the arrangement of main reinforcements and prestressed additional reinforcements in a corner of a reinforced concrete frame or arch; Figs. 2 and 3 show each a cross section through the web of a T-girder, the prestressed additional reinforcements being arranged in the lower portion or the corners of the web in which hair line cracks are most likely to form; Fig. 4 shows more schematically a method and means for preliminary or prestressing the additional bars or rods positioned in a form for molding a reinforced concrete beam; and Fig. 5 shows a cross section taken along a main reinforcement through a piece of concrete in which twisted or plaited, prestressed additional bars or wires are used to form a lattice work or netting.

Referring to Fig. 1, the corner portion of the mold or form in which the reinforced concrete structure forming a frame or arch is to be made is shown in cross section. Bent and otherwise suitably shaped main reinforcements or bars preferably of ordinary steel are shown therein in elevation. The position of additional preferably rod-like reinforcements of high quality steel which are to be prestressed is shown in dotted lines. Thus, additional reinforcements should be arranged in the horizontal planes shown in dotted lines *a—a* near the bottom and *b—b* near the top of the girder or beam forming the upper part of the frame or arch. Other additional reinforcements should be arranged in the plane *d—d'* in the upright forming the lateral part of the frame or arch. Additional reinforcements may also be arranged in the plane *c—c* across the corner, the additional reinforcements thus shown resulting in a polygonal arrangement composed of groups of additional reinforcements arranged in different planes. Removable abutments on the mold can be provided for prestressing the additional reinforcements, as indicated in Fig. 1; the prestressed reinforcements may also be anchored in the foundation, or wherever it is suitable. After the main reinforcements or rods have been placed in the mold or form and the additional preferably rod-like reinforcements uniformly prestressed, the concrete is poured or pressed into the mold, tightly around all those reinforcements, and allowed to set and shrink, whereby a tight and firm bond between the concrete and all the reinforcements is effected.

After the prestressed additional reinforcements have thus been firmly bonded with the concrete so that the prestress applied to them is maintained in them by that bond, the removable means, if any, for prestressing them relative to the form or mold, as well as the latter, can be removed.

The main reinforcements preferably consist of rods of ordinary steel as heretofore used for this purpose and have in any case a lower yield point and tensile strength than the additional reinforcements.

The total area of the cross sections of the main reinforcements is F_e^I (also designated with A_e^I); the total area of cross sections of the additional reinforcements is F_e^{II} (also designated with A_e^{II}). The total area of all the reinforcements, main and additional, $F_e = F_e^I + F_e^{II}$.

The main reinforcements have a yield point of f_y^I which is considerably lower than the yield point f_y^{II} of the additional reinforcements. Calling the average yield point of the two kinds of steel used for the main and additional reinforcements f_y^{III}, prestressing of the additional reinforcements should be effected to such an extent at the maximum that desired safety, and preferably double safety is obtained with respect to that average yield point f_y^{III}.

Although the additional reinforcements also add to the overall strength of the reinforced structure and their presence and cross sections, etc., are to be duly taken into consideration in calculating the structure, it should be understood that the prestressing of the additional reinforcements in the main serves to reduce the width of the hair line cracks if and when they occur under predetermined maximum or permissible load, or to prevent their coming into existence.

As stated above, prestressing of reinforcements embedded in and thereby bonded with the set and shrunk concrete results in compressive stresses within the body of the concrete surrounding each individual prestressed reinforcement, the absolute amounts of those compressive stresses forming a maximum close to the reinforcement and quite rapidly decreasing with the distance from that prestressed reinforcement.

Referring to Fig. 2 it will be appreciated therefore that the compressive stresses imparted to the concrete body or layer surrounding each individual additional reinforcement indicated by F_e^{II} will reach an adjacent main reinforcement indicated by cross hatching and reference character F_e^I, it being understood that the number of those additional reinforcements and their distance from the main reinforcements be such that this result can be obtained. Therefore, if in the concrete of that cross section tensile stresses are produced by a live load, they will be counteracted by those compressive stresses, and if the tensile stresses thus imparted to that cross section are greater than the compressive stresses and therefore eventually result in hair line cracks, the width of the latter will be considerably smaller than it would be if no prestressed additional reinforcements were present. Hence it follows that by the use of main reinforcements and a relatively small number of additional prestressed reinforcements of a steel of higher quality than the main reinforcements, furthermore by arranging the prestressed additional reinforcements in suitable proximity to the main reinforcements so that the compressive stresses are transmitted up to the main reinforcements, and finally by arranging the prestressed additional reinforcements in regions of the cross section and in suitable relation to a surface most endangered by tensile stresses caused therein by the load, the formation of hair line cracks of dangerous or undesirable size and width can successfully be prevented. The number of additional reinforcements and the extent of their prestressing can easily be calculated or ascertained by a few experiments in each individual case, and the maximum preliminary stress put on the additional reinforcements should be such that an average double safety or any other desired or prescribed safety of the structure is secured.

It will furthermore be appreciated from the showings of Figs. 2, 3 and 5 that the cross section of the individual rod-like main reinforcements is advantageously considerably larger than that of the additional rod-like reinforcements and consequently the total area F_e^I will very considerably exceed the total area F_e^{II}.

The load, dead as well as live, will therefore be carried in the main by the main reinforcements. As to the prevention of dangerous crack formation, or reduction of the size of the cracks formed under live load, to a desired extent, the prestressing of the additional reinforcements comes decisively into play.

Besides the number of individual additional reinforcements, also their tensile strength and yield point has to be taken into consideration.

Corners of reinforced concrete structures are particularly subject to hair line cracking and should be reinforced in the way to be derived from Fig. 3 without further comment.

Though anybody skilled in the art is capable of applying the invention in practice, it should be mentioned that according to tests carried out by the inventor, high quality steels can be successfully used for additional reinforcements having a safe tensile strength of 1000 to 1200 kg./sq. cm., this value to be taken as a basis for calculation of their permissible prestressing.

Most surprisingly, the tensile strength of the concrete, as small as it may be, can also be taken into consideration in calculating the structure according to the invention. With normal concrete of present time manufacture about 20 to 30 kg./sq. cm. tensile strength can be assumed for a concrete mass undergoing bending by the live or dead load. Due to the compressive stresses imparted to the concrete according to the invention, tensile strengths up to 50 and 75 kg./sq. cm. can safely be made a basis for calculation; in other words, if tensile stresses imparted to the most stressed zone of a structure by the dead and live load do not exceed 50 to 75 kg. per sq. cm., the concrete will neither loosen from the reinforcements, nor crack. Now, in calculating the structure, its cross section has to be taken into consideration in which the largest tensile stress occurs under predetermined maximum load. It is common to consider in such case only the total tensile strength of the reinforcements positioned in the stressed area of that cross section. If however the tensile stress per sq. cm. in that area does not exceed 50 to 75 kg., the tensile strength of the concrete in that area can be taken into consideration in addition to that of the total tensile strength of the reinforcements, main and additional, arranged in that area. If however calculation shows that the tensile stresses in the last mentioned area exceed that maximum value per unit of area, crack formation will occur, and consequently the old rule of basing the calculation only on the total tensile strength of all the reinforcements positioned in that stressed area of the cross section should be applied.

Heretofore hair line cracking was permitted and by and by the cracks extended from the outer surface of the stressed portion of the structure to its neutral axis. They gaped widely and thus admitted moisture into the interior of the structure which corroded the reinforcements and weakened the concrete. By arranging additional reinforcements according to the invention, particularly in the endangered zones of the most stressed cross section and near its outer surface, such dangerously gaping cracks can either be prevented or their width under predetermined load be reduced. It is a preferred feature of the invention to measure the degree of prestress on the additional reinforcements, furthermore to choose their number and distance from the main reinforcements so that under dead load the compressive forces produced by the prestressed additional reinforcements suffice to substantially close the cracks when formed under predetermined maximum live load.

Since the compressive stresses produced in the body of the concrete closely surrounding the prestressed additional reinforcements quasi improve the properties of that concrete in the way described above, it is even possible to use for the purposes of the invention concrete of lower quality than heretofore used for structures of same dimensions, purposes and loads. On the other hand, with that improvement in quality of the concrete according to the invention, reinforced concrete can be used for purposes heretofore unknown or only to be served by more complicated and expensive structures.

The degree of prestressing of the additional reinforcements depends upon the result to be obtained and particularly the maximum loads to be carried. In many structures and particularly those in which the live load is relatively small compared with the dead load, a low degree of prestressing suffices, and in such cases also the quality of steel used for the additional reinforcements need not considerably exceed that used for the main reinforcements. Since the individual additional reinforcements are of considerably smaller cross section than the individual main reinforcements, wires can be used for the former. Due to the manufacture of wires, their tensile strength and yield point is usually higher than those properties of the bars from which the wires have been drawn. Therefore, if relatively low degrees of prestress are to be applied, it suffices sometimes to use for the main reinforcements thicker rods and for the additional reinforcements drawn wires of substantially the same steel quality.

As pointed out above, all the reinforcements are placed in the mold or form, and prestresses are applied before the concrete is poured into the mold or form. However, it is within the realm of the invention to place all the reinforcements in the mold and to apply the prestressing after the concrete has been placed in the mold, it being understood that such prestressing has to be performed before the concrete starts to set.

Any means and method for prestressing the additional reinforcements can be applied.

Fig. 4 illustrates schematically the manufacture of a beam or girder in situ or in the plant. Only the additional reinforcements are shown, placed in the mold, and wires attached to the ends of these reinforcements and passed through the front walls of the mold to form a loop. At least within one of these loops a wedge *k* is positioned so that by driving it in, any desired degree of prestress can be applied to the additional reinforcements. After the concrete has been poured into the mold and finally set, the wedge or wedges are removed, the loops cut off, the mold is removed, and any still projecting ends of the pulling wires cut off. Instead of wedges, any other means for accurately effecting prestressing can be used, such as tensometers or expansion meters; also winches and screws can be used stressing the loop in Fig. 4, and by counting the number of revolutions of the screw the accurate stress desired can be applied.

In order to improve the bond between the concrete and the prestressed additional reinforcements, any means known in the art can be used. In particular, the additional reinforcements substantially arranged in the direction of the tensile stresses produced in the concrete by the load may be made of two or more twisted or plaited wires, as shown in Fig. 5, where the upper transverse wire illustrates a main reinforcement and the two plaited wires below an additional prestressed reinforcement. By arranging transverse wires passing the loops of the plaits as shown in Fig. 5, a lattice work or netting of additional reinforcement can be obtained. The transverse wires can be rigidly connected with the plaits, for instance by welding. Such transverse wires at the end of the plaits can then be formed into loops and used for prestressing the plaits in a method as described with reference to Fig. 4.

It will be appreciated from the above that by the invention many outstanding advantages are obtained as to saving of expensive high quality steel, more uniformity and ease of prestressing the relatively small number of additional reinforcements, improvement of the properties of the concrete used, preventing the formation or reducing the size and particularly avoiding dangerous widths of cracks, and prolonging the life of the final structure.

It should be understood that the invention is not limited to any exemplification or illustration herein shown, but to be derived in its broadest aspect from the appended claims.

I claim:

1. A beam, girder or frame structure, or part thereof, substantially consisting of concrete reinforced in the region of tension by rod-like reinforcements of steel and other rod-like reinforcements of superior tensile strength than and supplementing said first mentioned reinforcements, said other reinforcements arranged such distances from said first mentioned reinforcements and being prestressed so that the compressive stresses produced in the body of the concrete bonded with an individual other reinforcement extend to an adjacent main reinforcement.

2. A beam, girder or frame structure, or part thereof, substantially consisting of concrete reinforced in the region of tension by rod-like reinforcements of steel and other rod-like reinforcements of superior tensile strength than and supplementing said first mentioned reinforcements, said other reinforcements arranged such distances from said first mentioned reinforcement and a surface under tension of the structure and being prestressed so that the compressive stresses produced in the body of the concrete bonded with an individual other reinforcement extend to an adjacent first mentioned reinforcement and said surface, thereby substantially reducing or preventing hair line cracking of the concrete.

3. A beam, girder or frame structure, or part thereof, substantially consisting of concrete reinforced in the region of tension by rod-like main reinforcements of steel and other rod-like reinforcements of superior tensile strength than and supplementing said main reinforcements, said other reinforcements being sufficiently prestressed and arranged such distances from a surface under tension of the structure subject to hair line cracking under live load so as to keep, by the compressive stresses produced in the concrete by said prestressed other reinforcements, within a predetermined limit the width of such cracks when formed under predetermined maximum live load and to sustantially close those cracks when the live load is removed.

4. A beam, girder or frame structure, or part thereof, substantially consisting of concrete reinforced in the region of tension by rod-like main reinforcements of steel and other rod-like reinforcements of superior tensile strength than said main reinforcements, said other reinforcements being prestressed and arranged such distances from said main reinforcements as to substantially improve by the thus produced compressive stresses in the concrete the tensile strength of the concrete bonded with them and supplement said main reinforcements.

5. A beam, girder or frame structure, or part thereof, substantially consisting of concrete reinforced in the region of tension by rod-like main reinforcements of steel and other rod-like reinforcements of superior tensile strength than said main reinforcements, said other reinforcements being prestressed and arranged such distances from said main reinforcements and a surface under tension of the structure so as to improve by the thus produced compressive stresses in the concrete the tensile strength of the concrete bonded with them, supplement said main reinforcements and substantially reduce or prevent hair line cracking of the concrete.

6. A beam, girder or frame structure, or part thereof, substantially consisting of concrete reinforced in the region of tension by main rods of steel and other rod-like reinforcements of superior tensile strength than and supplementing said main rods, said other reinforcements exemplified by steel wires being of considerably smaller cross section than said main rods, said other reinforcements being prestressed and arranged such distances from said main rods and a surface under tension of the structure so as to substantially reduce or prevent hair line cracking of the concrete by the compressive stresses produced in the concrete bonded with said prestressed other reinforcements.

7. A beam, girder or frame structure, or part thereof, substantially consisting of concrete reinforced in the region of tension by main rods of steel and other rod-like reinforcements of superior tensile strength than and supplement-

ing said main rods, said other reinforcements being of substantially smaller cross section than said main rods and combined in networks as exemplified by plaited strands extending substantially in the direction of tensile stresses produced in the concrete under load and wires interlaced with said strands, said wires extending in a substantially vertical direction to said strands, said other reinforcements being sufficiently prestressed and arranged such distances from said main rods and a surface under tension of the structure as to substantially reduce or prevent hair line cracking of the concrete by the compressive stresses thus produced in the concrete and substantially extending to said adjacent main reinforcements and surface.

FRITZ EMPERGER.

A.1.5 Brit. 554,693

AMENDED SPECIFICATION

Reprinted as amended in accordance with the Decision of the Superintending Examiner acting for the Comptroller-General dated the first day of April, 1958, under Section 29, of the Patents Act, 1949.

PATENT SPECIFICATION

554,693

Application Date: Jan. 12, 1942. *No. 430/42.*

Complete Specification Left: Jan. 12, 1943.

Complete Specification Accepted: July 15, 1943.

PROVISIONAL SPECIFICATION

Improvements in Reinforced Concrete Structures and in the manufacture thereof

I, Paul William Abeles, of 12, Royal Crescent, London, W.11, of Austrian nationality, do hereby declare the nature of this invention to be as follows:—

Reinforced concrete structures have previously been proposed in which reinforcing members of mild or middle grade steel, distributed in the concrete, are subjected to an initial tension. Since the latter has to be below the limit of elasticity the initial tensile stress is considerably reduced owing to the elastic deformation of the concrete at releasing of the stretching force upon the concrete, shrinkage and creep, thus only a fraction of the initial tension remains, when under load. All the advantages of tensioning are therefore lost.

In specifications Nos. 338,864 and 338,934 an improvement has been proposed by using reinforcing members of high tensile steel to be subjected to an initial tensile stress substantially greater than the limit of elasticity of steel as normally used, the tension being applied either before setting and hardening or during or after hardening, the stretching force applied being of such magnitude that after releasing it upon the concrete and after shrinkage and creep have taken place no cracks arise, when under load, thus a material is obtained which behaves like a homogeneous one. In these specifications it has further been proposed in such structures to employ additional unstretched reinforcing members in combination with reinforcing members of high tensile steel under tension. In such a construction considered as behaving like one of homogeneous material, when under load, only compressive stresses result over the whole cross-section, or in some cases tensile stresses which, however, must be substantially less than the tensile strength of the concrete in order to ensure that no cracks arise. In structures according to these specifications it has been necessary to apply to such tensioned members a very great stretching force in order to ensure the homogeneity of the structure, when loaded. The greatest compressive stresses arise in the structure, not at loading, but at releasing the stretching force on the concrete, thus special methods are necessary to ensure a rapid hardening of the concrete to great strength which is especially important in cases where the tension is applied prior to the hardening of the concrete.

In specification No. 541,835 has the inventor proposed another solution relating to reinforced concrete structures which comprise bars or rods, preferably of high tensile steel, employed in the normal manner, in combination with thin wires of high strength to which an initial tension is applied to cause a counter-bending moment in the structure and exert a pressure upon the concrete and upon the untensioned bars or rods, the stretching force applied to the wires being relatively considerably smaller than that necessary in the prior construction above referred to, the necessary safety factor against breaking of the structure being ensured, the reinforced concrete structure according to that invention being such that if regarded for the purpose of calculation as made of a homogeneous material, a tensile stress would result in the concrete greater than its tensile strength. That improved structure thus differs from the prior proposed construction in that it is not designed to behave as if made of a homogeneous material, and when loaded may allow the formation of cracks in the tensile zone of the concrete the width of which may be of such order as regarded as permissible in normal structures in which no initial tension is applied to the reinforcing members. A further distinction of importance between the prior construction and that of this invention is that the magnitude of the stretching force to be applied to the reinforcing members is considerably less than that applied to the tensioned members previously proposed. By the use of thin wires of high strength and relatively small cross-sectional areas for the tension at reinforcement, in relatively simple forms of anchorage are found to be adequate. The said wires may be employed singly or in strands comprising several wires.

The present invention deals with reinforced concrete structures with tensioned reinforcement of high strength, the applied stretching force in the same way as with the previous invention specification 541,835 being only of such magnitude that in the cross-section of the structure, when calculated as of a homogeneous material, the greatest resultant tensile stresses under load are in excess of the tensile strength of the concrete. According to the present invention the tensioned tensile reinforcement comprises bars or rods of high tensile steel or wires or cables of high strength or a combination of them. The stretching force is similar to that according to specification 541,835 greatly reduced compared with the stretching force necessary in case homogeneity is ensured, thus the average stretching stress according to the present invention (which may be applied to the single reinforcing members in the same or different degree) is smaller than according to the inventions described above, but greater than the limit of elasticity of mild steel. The tensioning may be applied before or after hardening of the concrete. Provisions may be made in order to reduce the magnitude of the elastic deformation, shrinkage and creep by any known specific mixture and/or curing of the concrete, the reduction of the initial prestress thereby being minimised.

In cases where the tension is applied to the reinforcement after hardening of the concrete anchoring devices are required but in cases where the tension is applied to the reinforcement before hardening of the concrete anchoring devices may be dispensed with owing to the relatively large surface areas of high strength reinforcing members in comparison with their cross-section, and the consequent high strength of the bond with the concrete surrounding the same. When the tension is applied to the reinforcing members after hardening of the concrete, in order to avoid the difficulty caused by the bonding of the reinforcement in the concrete, which would result in the tension acting only upon the ends of the reinforcing members leaving the central portions thereof unstressed, the reinforcing members may be prevented from bonding with their surrounding concrete by being coated with a yielding substance such as asphalt or bitumen or in any other convenient manner, for example by being laid within thin pipes within the concrete, or by tapes of paper or the like being wound around the reinforcing members. By these means it can be ensured that the tensioning force applied to the ends of the reinforcing members results in an equal tension throughout the length of them.

The reinforcing members may also be threaded through a tube hole and afterwards tensioned. Such a reinforced concrete construction according to the present invention may also consist of a plurality of single pre-cast units with holes through which the reinforcing members are threaded, the anchorages against the end units being carried out after tensioning in any suitable way, the joints between the units may have interlocking devices in order to take up the shearing straining. Beside the ties at the tension zone also connections at the pressure zone may be provided.

The stretching process may be performed in a similar way to the alternatives described in specification 541,835 or in any other suitable way. In case of pre-cast reinforced concrete structures, the tensioning of the reinforcement being applied before hardening, a plurality of units may be arranged either in a continuous series laid coaxially or one beside the other, or one above the other, a combination of two or all three of these possibilities being applicable. In such a case, the tensioned members are anchored at one end and tensioned at the other end, the releasing of the stretching force being effected by cutting the reinforcing members.

Dated the 12th day of January, 1942.

PAUL WILLIAM ABELES

COMPLETE SPECIFICATION

Improvements in Reinforced Concrete Structures and in the manufacture thereof

I, Paul William Abeles, Dr. of Eng., of 12 Royal Crescent, London, W.11, of Austrian nationality, do hereby declare the nature of this invention and in what manner the same is to be performed, to be particularly described and ascertained in and by the following statement:—

This invention relates to improvements in reinforced concrete constructions of the kind in which the reinforcement is "pre-stressed" (i.e. tensioned before the load is applied), such reinforcement consisting of high strength steel or wire, the average initial pre-stress being greater than the elastic limit of mild steel.

In the case of tensioning before setting of the concrete ("pre-stretching") on the release of the stretching force, applied to the reinforcement, the concrete is placed in compression, but the pre-stress is reduced due to the immediate elastic deformation and shrinkage of the concrete, these losses of the stress in the reinforcement being further increased by the further gradual shrinkage and creep of the concrete. The sum of these losses amounts to a stress not much less than, or even in excess of, the elastic limit of mild steel. In cases where the stretching force is transmitted to the concrete after its setting and hardening ("post-stretching") the losses of the initial pre-stress are reduced to those occurring due to gradual shrinkage and creep of the concrete. But even in this latter case where the concrete has hardened before the stretching force is released, the total losses of the pre-stress may be not much less than the elastic limit of mild steel. The initial pre-stress has therefore to be considerably greater than the elastic limit of mild steel if it is to be of value and effective in the finished structure under load conditions.

If the stretching force is of such a magnitude that, when in use, even after all losses of the pre-stress have taken place, the concrete sections remain under permanent compressive stresses, concrete tensile stresses being totallv excluded, cracklessness can then be guaranteed and a product is obtained which behaves in a similar manner to one formed of homogeneous material. In order to ensure such a stress-distribution, with the exclusion of any concrete tensile stress, a stretching force has to be applied to the reinforcement equal to, or in excess of, a magnitude which can be exactly computed for the limit of a straight line stress-distribution with the stress zero at the outside tensile fibre when the structure is under the maximum load for which it is designed. Such a tensioning is hereinafter referred to as "fully" pre-stressing.

In specifications Nos. 338,864 and 338,934 "full" pre-stressing has been proposed by using reinforcing members of high tensile steel, subjected to an initial tension substantially greater than the elastic limit of steel as normally used in reinforced concrete, whereby after all shrinking of the concrete due to setting and hardening has taken place, the concrete remains under permanent compressive stresses. The subject of these inventions is the creation of a homogeneous material having entirely new, important and surprising properties, imparted to articles made in accordance therewith by the pre-stressing of the reinforcement and thus ensuring cracklessness.

In structures according to these specifications it has been necessary to apply to such tensioned reinforcing members a very great stretching force in order to fulfil the condition of "full" pre-stressing. The greatest compressive stresses occur in the structure, not at loading, but upon release of the stretching force on the concrete, thus necessitating a great strength already in existence before transmitting the stretching force to the concrete.

According to the present invention in a pre-stressed reinforced concrete structure, intended in use to be mainly strained by bending the tensile reinforcing members, which are tensioned, comprise bars, rods, wires or cables, of an elastic limit higher than that of mild steel, or a combination of such members, the individual members being of the same strength properties or of different strength properties, the said reinforcing members being pre-stressed either before or after hardening of the concrete, or partly before and partly after hardening, the initial pre-stress of the individual members either being equal or different but the average initial pre-stress being greater than the elastic limit of mild steel, and the total initial stretching force, applied to the reinforcing members, being of such a magnitude that, when bending of the structure takes place under the maximum load for which the structure is designed, a straining in the most unfavourably stressed cross section of the structure occurs which corresponds to a straight line stress distribution for a homogeneous material, having a resultant concrete tensile stress in excess to a permissible extent of the concrete Tensile strength, ascertained or assessed in the manner hereinafter described, taking into account the greatest possible losses of the pre-stress. By "to a permissible extent" is meant to an extent such that the cracks which may occur are not of greater width than is normal in ordinary reinforced concrete structures with unstressed reinforcement.

The invention is distinguished from "full" pre-stressing, as hereinabove referred to, in that the resultant concrete stress at the tensile fibre under maximum load is a "tensile" stress. Homogeneity can, however, be ensured even if there are tensile stresses in the section, provided such stresses are considerably less than the tensile strength of the concrete. According to the present invention the greatest stress at the tensile fibre is "in excess of the concrete tensile strength", thus further differentiating between "full" pre-stressing, where the total absence of cracks is guaranteed, and "partial" pre-stressing, employed in this invention, where complete absence of cracks and homogeneity cannot be guaranteed.

The stretching force applied to the reinforcement according to the present invention is considerably smaller than that employed in fully prestressed articles.

The use of high strength steel as reinforcement where the elastic limit of such steel was greater than one and a half times that of mild steel, has previously been considered to be permissible without increasing the factor of safety against breaking, only if it is fully prestressed, because of the danger of heavy cracks due to the great elongation of the steel in the case of high stresses. Tests have proved that this restriction is not necessary, and that with a partial pre-stretching of the reinforcement the occurrence of heavy cracks can be avoided. It is therefore not necessary to tension the reinforcement to the extent required for full pre-stressing, thus making possible a reduction in the cost of performing the stretching process. Moreover, the stresses, set up in the concrete at the releasing of the pre-stress, are greatly reduced, which represents another advantage of this invention as compared with fully pre-stressing.

The cracks which occur in structures according to the invention when under the maximum designed load are not dangerous when the whole of the reinforcement is "pre-stretched" or only a part thereof is "post-stretched" for where cracks occur, the bond between the concrete and the whole or a part of the reinforcement respectively is only destroyed in the immediate neighbourhood of the cracks, and therefore elongation of the reinforcement is confined to the very small part thereof extending across the cracks, thus confining their extent. With structures in which the whole of the reinforcement is "post-stretched", although there is no bond between the concrete and the reinforcement, and thus a relatively greater elongation takes place, causing exposure of a part of the reinforcement with consequent risk of rusting thereof, even such cracks do not affect the safety of the structure, if precautions are taken to prevent such rusting as hereafter more fully described.

In my previous specification No. 541,835 I have proposed another form of "partial" pre-stressing, based upon similar considerations and with a similar stress distribution under working load. The invention disclosed in my said specification No. 541,835 relates to structures in which a part of the tensile reinforcement is either pre-stretched or post-stretched and a part remains unstretched, whereas the present invention relates to reinforced concrete structures in which the whole of the tensile reinforcement is pre-stressed, either by pre-stretching, or post-stretching, or by partly pre-stretching and partly post-stretching, the initial stretching stress in the reinforcement being the same for each reinforcing member or varying in different reinforcing members but in any case the average stretching stress, (i.e. the total stretching force divided by the total area of the reinforcement) being greater than the elastic limit of mild steel.

In structures according to the present invention, when compared with structures having unstretched reinforcement, the steel consumption can be greatly reduced, for instance, the tensile reinforcement can be safely reduced to about 1/5 of the usual weight of mild steel, without reduction of the factor of safety against breaking and without the occurrence of cracks of greater width than are normal in unstressed reinforced concrete. Compared with unstressed reinforced concrete structures another advantage is attained by the fact that structures according to the present invention in fact behave as if made from a homogeneous material, when they are unloaded or only slightly loaded. This means that cracks which may occur at the stage of full loading close totally, when the superimposed load is greatly reduced or removed, thus reducing the danger of rusting of the reinforcement. Due to the pre-compression of the concrete, effected by the stretching force, the permanent deformation of the structures after cracks have occurred is greatly reduced as compared with normal reinforced concrete structures.

The stretching process may be applied to the reinforcing members in any known way. In the case of prestretching, the stretching force is transmitted to the concrete preferably by bond. In the case of precast reinforced concrete structures, the tensioning of the reinforcement being applied before hardening of the concrete, a number of units may be arranged either in a continuous series laid co-axially or one beside another, or one above the other, a combination of two or all three of these arrangements being possible. In such a case the tensioned members are anchored at one end and the stretching force applied at the other end, the releasing of the stretching force being effected by cutting the reinforcing members, or by gradual reduction of the stretching force at the end, where applied.

In the case of post-stretching, the reinforcing members are prevented from bonding with the concrete in any convenient manner, e.g. by leaving upon the surface of the members the grease normally found thereon in commerce (thus also preventing rusting at wider cracks, which may occur when the whole of the reinforcement is post-stretched) by applying a lubricant to the surface of the reinforcing members, by paper wound around the reinforcing members by providing thin pipes preferably also of high strength steel in which the reinforcing members are laid, which pipes may also be pre-stressed, thus being used as reinforcement, or by any other known method. The stretching process may be carried out either by nuts engaging threaded parts of the reinforcement by twisting thin wires together, or by anchoring one end of the reinforcing members and applying a tensioning force to the other end thereof by any known tensioning means. The resulting compression is transmitted to the concrete by anchor plates at the ends of the reinforcements, which anchor plates are advantageously provided with jaws to grip the tensioned reinforcement which is after-cut.

A reinforced concrete structure according to the invention may also consist of a plurality a single prefabricated elements of concrete, cement, light concrete, brick, clay or any other material moulded from a plastic substance, these single elements being solid or hollow, in either case being prefabricated with apertures through which the reinforcing members are threaded, the anchorage against the end elements being carried out after tensioning of the reinforcement in any suitable way, the joints between the elements comprising interlocking parts, if desired, in order to take up shear stresses. In addition to the tensile reinforcement, which serves as ties, a similar reinforcement in the pressure zone may be provided. A film of grease may be left on the surface of the reinforcing members to avoid rusting at the joints, where gaps may occur between the single elements.

The invention will now be described with reference to the accompanying drawings, in which

Fig. 1 is a section through a rectangular reinforced concrete beam according to this invention, and

Figs. 2 to 7 are stress diagrams, illustrating the invention in comparison with previously known concrete structures with unstretched mild steel reinforcement and with fully prestressed structures.

As shown in Fig. 1, a rectangular reinforced concrete beam 1 is provided with reinforcing members 2 at the tensile zone and preferably also with reinforcing members 3 at the compressive zone. The reinforcing members 2 are pre-stressed, the initial tensioning stress of the individual reinforcing members either being equal or different. The reinforcing members 3 serve as reinforcement for the counterbending caused by the pre-stress in the members 2, and these members 3 may also be prestressed. This is necessary if the structure has to withstand straining in two or more directions, for instance in the case of poles or stanchions of frames, subject to wind pressure, in which case the members 3 (as well as the members 2) constitute tensile reinforcement.

Figs. 2 and 3 show the stress-distribution in the section due to the pre-stress on the basis of a straight line stress-distribution for a homogeneous material after reduction of the pre-stress owing to all the losses previously mentioned (due to elastic deformation of the concrete, shrinkage and creep). Fig. 2 represents the case where the reinforcing members 2 only are pre-stressed, a concrete tensile stress occurring at the upper fibre and a compressive stress at the lower fibre, contrary to the straining at loading. Fig. 3 illustrates the stress-distribution if both the reinforcing members 2 and 3 are pre-stressed, only compressive stresses occurring, if the stretching force applied to 3 is in excess of a given limit, which can easily be computed. It is, of course, possible to stretch the reinforcing members to such an extent that the compressive stress is of the same magnitude all over the whole section. At the stage of transmitting the stretching force to the concrete the stresses are, of course, greater than those shown in Figs. 2 and 3, since the pre-stress is not then reduced owing to creep and owing to that part of shrinkage which takes place after releasing.

Fig. 4 illustrates the stress-distribution due to loading alone on the basis of a straight line stress-distribution for a homogeneous material assuming the reinforcements 2 and 3 to be of equal area. The usual stress-distribution for the design of a concrete section with unstretched reinforcement is illustrated in Fig. 5, the concrete tensile zone being neglected and the whole tensile force T taken up by the reinforcement being balanced by the compressive force C.

In the case of "full" pre-stressing, the stretching force or forces have to be of such a magnitude or of such magnitudes that the resultant stress-distribution due to maximum loading (according to Fig. 4) and due to prestressing (according to Figs. 2 or 3) is in accordance with that illustrated in Figure 6 having only compressive stresses over the whole section.

Fig. 7 illustrates the resultant stress distribution in the case of "partial" pre-stressing, tensile stresses in the section not being excluded as with "full" pre-stressing (Fig. 6).

The concrete Tensile Strength, referred to hereinabove, may be ascertained from test specimens (for example from cylinders or briquettes). Where the concrete Tensile Strength has not been ascertained from special test specimens, it may be assessed from the known compressive Cube Strength of the concrete. For various qualities of concrete the ratio between the Cube and the Tensile Strength varies greatly. The exact strength of the concrete is not of major importance, it being sufficient to consider a weak concrete of a cube strength up to 3000 lb. per sq. in. as having a tensile strength of up to 200 lb. per sq. in., a middle-strength concrete of a cube strength between 3000 and 6000 lb. per sq. in. as having a tensile strength of between 200 and 350 lb. per sq. in. and a high-strength concrete of a cube strength between 6000 and 10,000 lb. per sq. in. as having a tensile strength of between 350 and 500 lb. per sq. in. (over 10,000 lb. per sq. in. the tensile strength may be regarded as approximately 1/20 of the cube strength).

Hereinbefore, and in the claims the expression "concrete" is intended to mean a product characterised by the fact that it is moulded from a plastic substance, consisting of a mixture of natural or artificial, preferably suitably graded, aggregates of light or heavy weight, and of a binding agent, the resulting product being any of those known as concrete, artificial stone, artificial lime-stone, synthetic concrete, dry clay, dry earthenware or the like.

In the foregoing description, the invention has been described with reference to structures such as beams of a rectangular cross section, but it is to be understood that the invention is equally applicable to reinforced concrete structures of other cross sectional shapes, such as T-shape or I-shape, and to reinforced concrete structures generally.

Having now particularly described and ascertained the nature of my said invention, and in what manner the same is to be performed, I declare that what I claim is:—

1. A pre-stressed reinforced concrete structure, intended to be used mainly strained by bending, with tensioned tensile reinforcing members comprising bars, rods, wires or cables, of an elastic limit higher than that of mild steel, or a combination of such members, the individual reinforcing members being of the same strength properties or of different strength properties, the initial pre-stress of the individual members either being equal or different, but the average initial pre-stress being greater than the elastic limit of mild steel, and the total initial stretching force, applied to the reinforcing members, being of such a magnitude that, when bending of the structure takes place under the maximum load for which the structure is designed, a straining in the most unfavourably stressed cross section of the structure occurs which corresponds to a straight line stress distribution for a homogeneous material, having a resultant concrete tensile stress in excess to a permissible extent of the concrete Tensile strength, ascertained or assessed in the manner hereinbefore described, taking into account the greatest possible losses of the pre-stress.

2. A pre-stressed reinforced concrete structure according to claim 1, wherein the tension is applied to the reinforcing members before the concrete is set, the pressure exerted upon the concrete being transmitted thereto by the bond between the concrete and reinforcing members.

3. A pre-stressed reinforced concrete structure according to claim 1, wherein the tension is applied to the reinforcing members after setting and hardening of the concrete, means being provided to prevent the formation of bond between the concrete and the reinforcing members, and means being further provided for anchoring the ends of the reinforcing members after the application of the tension.

4. A pre-stressed reinforced concrete structure according to claim 1, wherein the tension is applied to a part of the reinforcing members in accordance with claim 2, and to a part in accordance with claim 3.

5. A pre-stressed reinforced concrete structure according to claim 2, wherein a number of units are arranged either in a continuous series, laid co-axially, or one beside another, or one above the other, or a combination of two or all three possibilities, the reinforcing members being anchored at one end and tensioned at the other end, the transmitting of the stretching force to the concrete being effected by cutting the reinforcing members, spacing members being arranged between the single units.

6. A pre-stressed reinforced concrete structure according to claim 3, wherein the formation of bond is prevented by the provision of a film of grease upon the surface of the reinforcing member (such film also preventing rusting) or by paper tapes wound around the reinforced members.

7. A pre-stressed reinforced concrete structure according to claim 3, wherein the tensioning of the reinforcing members is carried out by nuts engaging the threaded ends of the reinforcing members against anchor plates.

8. A pre-stressed reinforced concrete structure according to claim 3, wherein the tensioning of the reinforced members is carried out by pulling devices, the said tensioned reinforcing members being gripped by jaws forming parts of anchor plates, thereafter being cut off adjacent to the gripping jaws.

9. A pre-stressed reinforced concrete structure according to claim 3, wherein the tensioning of the reinforcing members is carried out by pulling devices, and the tensioned reinforcing members are anchored by wedge pieces to prevent relaxation of the tension and thereafter cut above such wedge pieces.

10. A pre-stressed reinforced concrete strucsture according to claim 3, wherein the structure consists of a plurality of single prefabricated concrete elements, the single prefabricated elements being solid or hollow and having apertures through which the reinforcing members are threaded, and the said single elements having preferably interlocking parts at their joints to take up shear stresses, the reinforcing members, after being threaded through the said apertures and tensioned by a pulling device, being anchored against anchor plates, provided at the end elements.

11. A pre-stressed reinforced concrete structure according to claim 4, wherein the part of the reinforcement which is pre-stretched comprises thin pipes of high strength steel and the post-stretched part of the reinforcing members is arranged within said pipes.

Dated the 12th day of January, 1943.

PAUL WILLIAM ABELES

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Fig. 1.

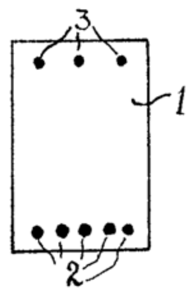

Fig. 2.

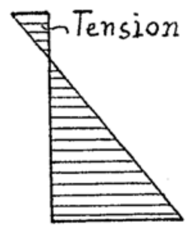

Fig. 3.

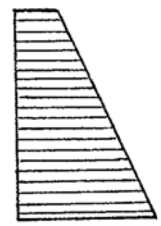

Fig. 4.

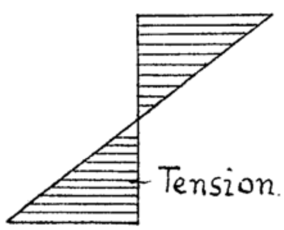

Fig. 5.

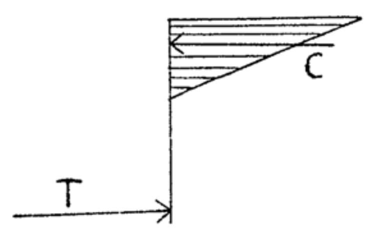

Fig. 6.

Fig. 7.