Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads*

by PCI Committee on Precast Concrete Bearing Wall Buildings

Comments by Emil C. Hach, William Hanuschak, Alan H. Mattock and Committee

EMIL C. HACH†

The Committee and Mr. Speyer are to be commended for a very valuable report. It would appear that much if not most of the report deals with good construction irrespective of the type of loads, i.e., normal or abnormal.

From the writer's vantage as a consulting structural engineer for a systems precast concrete bearing wall type building a few comments seem in order.

1. The spacing of expansion joints in general at 180 ft apart in Section 5.5 does not seem reasonable. We have seen problems in buildings under 150 ft and not seen them in buildings over 200 ft depending on the climate and layout of the bearing walls. While it would be nice if one could set a certain number down, it would appear that is not possible in a recommended practice.

Finally, the spacing of expansion joints is not dictated by abnormal loads. Rather, it is determined by the normal forces due to shrinkage, temperature and creep. In our view, this does not belong in this report.

2. In Section 7.8.3 the minimum amount of steel is set at $0.001bL$. Neither b nor L are defined here. Elsewhere (Sections 7.2.1 and 7.2.2) they are defined as the height and length, respectively, of a wall. This of course, is not what was meant. If it is the usual thickness times length (or width), then it is substantially less than that in ACI 318-71 either Chapter 14, 10 or 11. We question whether less reinforcing steel than that required in ACI 318 should be sanctioned for reinforced concrete precast bearing walls.

3. The requirement of Section 6.2.2 of 2½ in. and 3½ in., respectively, for bearing of solid/hollow-core slabs and ribbed slabs, respectively, is of some interest. About 50 buildings of the type we have been involved with have been built since "Operation Break Through." All have been built with solid slabs bearing on precast walls. Tolerances are as per the *PCI Design Handbook* for hollow-core slabs (p. 8-34). The bearing length of these solid slabs has been 2 in.

In questioning personnel involved with

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these buildings and from our own experience, no distress due to insufficient bearing has been noted. A check of bearing stresses shows that considerably less bearing than 2 in. is required before problems from insufficient bearing would be expected to occur.

We would expect some hollow-core slabs would have problems, but then this is covered in the beginning of the paragraph, "Precast floor or roof elements should have a sufficient bearing length to safely transfer applied loads by direct bearing." We would suggest that 2 in. for solid slabs is sufficient and the requirement should be changed to this.

WILLIAM HANUSCHAK*

The Committee (with Mr. Speyer's assistance) have presented practical minimum recommendations for the design of precast elements and particularly for connections to ensure a degree of continuity and stability to withstand abnormal loadings.

The writer believes that this report is welcomed by designers particularly for low to medium rise precast buildings in non-seismic areas where simple gravity type connections are used and these minimum requirements will govern.

The application of the "minimal tie requirements" between walls and floors as outlined in Chapter 6 should answer the concern many designers have had with such buildings and particularly their connections.

This PCI committee is to be commended for offering these guidelines to designers thereby ensuring a consistent and reasonable minimum standard and safety factor for precast bearing wall buildings.

ALAN H. MATTOCK†

Mr. Speyer is to be congratulated on assembling a most useful set of design guidelines for precast bearing wall buildings. The references given should also be very useful to any designer involved with this type of construction.

In Section 9.5.3, it is proposed that the shear-friction theory be used for the design of connections between the wall panels. For smooth joints, it is proposed that the shear-friction coefficient μ be taken to be 0.70 when designing the reinforcement. This value for μ was apparently taken from Table 6.1.3 of the *PCI Design Handbook*,[33] since Section 11.15—Shear Friction of ACI 318-71[1] only permits the shear-friction method of shear transfer design to be used when a roughened interface exists.

The origin of the value of 0.70 for μ given in Table 6.1.3 for a smooth concrete interface is uncertain, since no shear transfer test data existed for this condition at the time the *PCI Design Handbook* was written. It is probable that it was based on judgement, having in mind the value of μ established experimentally for a concrete-structural steel interface.

Tests recently made at the University of Washington of shear transfer across a concrete to concrete smooth interface indicate that the value of $\mu = 0.7$ is unconservative for this situation. The actual variation of shear transfer strength with amount of shear transfer reinforcement provided is shown in Fig. A. The shear strength is expressed as a nominal shear stress, $v_u = V_u/(bd)$; and the amount of reinforcement is expressed in terms of the reinforcement parameter:

$$\rho f_y = \frac{A_{vf} f_y}{bd}$$

In plotting the data and correlating it with the shear-friction theory, the value of the capacity reduction factor ϕ was taken as 1.0, since all material strengths and specimen dimensions were known exactly. It can be seen that the shear strength of a smooth concrete-to-concrete interface can be predicted closely using the shear-friction equation provided the value of μ is taken as 0.60.

In Series D, every precaution was taken to obtain a good bond between the precast concrete and the concrete cast

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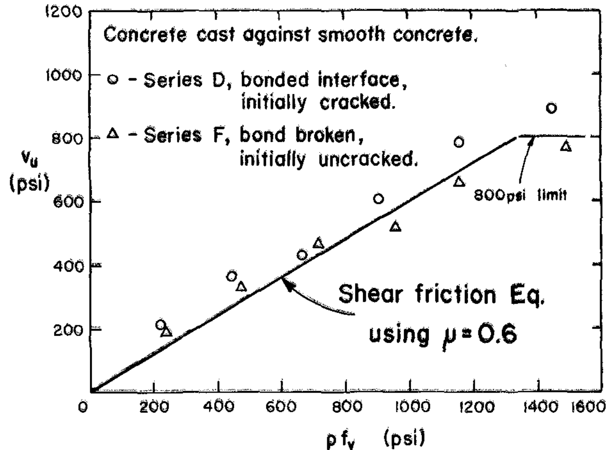

Fig. A. Shear strength of smooth concrete joints.

against it. A crack was then induced at the interface before making the shear test, by applying line loads to the front and rear faces of the specimens. This is so as to be consistent with the underlying philosophy of the shear-friction theory, which is that a crack exists in the shear plane before the shear acts.

It was also thought that the effect on shear strength of loss of bond at the interface should also be studied, so in Series F, bond at the interface was deliberately prevented by applying a bond breaker to the surface of the precast concrete before the other concrete was cast against it.

It can be seen that the shear strengths obtained for the two interface conditions were very similar and that a value of $\mu = 0.60$ is appropriate when using the shear-friction theory to calculate the shear strength of a smooth concrete to concrete joint.

The higher value of $\mu = 0.7$ obtained in tests of structural steel-concrete interfaces relates to the use of headed stud shear connectors as shear transfer reinforcement in that case. The higher value of μ obtained for the smooth steel-concrete interface than for the smooth concrete-concrete interface is due to the difference in the stress-strain curves between the headed stud shear connectors and the reinforcing bars used, and also due to the local enlargement of the stud cross section at the shear plane by the weld metal deposited around the circumference of the stud.

COMMITTEE CLOSURE

The Committee thanks Messrs. Alan H. Mattock, Emil C. Hach and William Hanuschak for their comments on these recommendations for precast concrete bearing wall buildings. The Committee appreciates Professor Mattock sharing with us research done on shear friction at the University of Washington. The report will be changed to reflect this recommendation of $\mu = 0.6$ for smooth concrete interface.

In reply to comments by Mr. Hach:

1. The Committee felt that spacing of expansion joints should be mentioned in the report so that engineers inexperienced in working with precast concrete would not think that, because the structure is composed of precast pieces, joints for thermal, creep and shrinkage control are not necessary. Creep of slabs due to prestressing also has an effect on the length between expansion joints. The consensus of the Committee was that in a straight line building setting a practical limit of 180 ft was consistent with previous experience in this type of structure.

2. The Committee felt that a lower mini-

mum reinforcement is justified for precast panels than for cast-in-place walls which governed ACI 318-71 code requirements. Precast panels have practically no restraint at the edges during the curing and storing stages and, therefore, will not build up tensile shrinkage stresses as high as those in cast-in-place walls. For lightly loaded interior walls the reinforcement may be placed along the periphery only (Reference 3, p. 12).

3. To insure the strength requirements of the grout column between slabs where analysis requires a grout column, and at the same time, minimum support dimensions for slabs, the Committee is now recommending minimum plan dimension of 2 in. for 6-in. thick and 2.5 in. for 8-in. thick interior walls. The Committee recognizes that in many structures considerably less bearing than this would not cause distress at this connection.

The Portland Cement Association, under a contract to the U.S. Department of Housing and Urban Development (H-2131R), is conducting an analytical study and large scale testing of elements of large-panel concrete structures. Results to date from this research show no serious deficiencies in this Committee report.[35-38]

The PCA test series on multistory wall panel assemblies (not yet published) have shown that assemblies composed of precast wall elements and untensioned strand as reinforcement placed within the horizontal joint at every story can be detailed to perform as a cantilever (when a wall panel is ineffective) in a monolithic and ductile manner.

A second PCA test series on slab systems of large panel structures (not yet published) has demonstrated the need for large slab distortions (sag) without collapse. This is achieved by placing short lengths of untensioned strand as reinforcing in the key joints over the support, and designing for bond movement prior to tension failure of the strand.

A third PCA test series on vertical load-carrying capacity of horizontal connections of large panel structures is still in progress.

The Committee will continue its liaison with PCA on this important research and provide industry review and input to PCA.

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The Editors welcome discussions of papers published in the PCI JOURNAL. The comments must be confined to the scope of the paper under discussion. Please note that discussion of papers appearing in this current issue must be received at PCI Headquarters by November 1, 1977.

T. Y. Lin Symposium on Prestressed Concrete*

A Discussion in Verse of the Special Commemorative Issue of the T. Y. Lin Symposium on Prestressed Concrete

by SHU-TIEN LI†

There came another from T. Y. Lin, with affection,
His Symposium issue and transpiration,
Nothing more fitting than this commemoration
Of his so many a lasting contribution.
We noted his talent before graduation,
While I was president at his institution.
At 20, he won *summa cum laude* distinction,
His post achievements give gratification.
Pages of Dr. P.C. just records my reflection.
Now, I add in discussion via verse narration:
By giving marble-iron an interaction,
Ancients made wonders for their utilization,
In 24 centuries ago, in the construction
At A-crop-o-lis gateway installation,
Enabling marble blocks bearing long-span action,
Marking the then Hellenic civilization.
Modern high-strength concrete and steel production
Has given prestressing new acceleration.
For the last only half-century duration,
It has risen to dominating position.
In an age of materials integration,
There should be no institute's separation.
May we learn from materials combination,
To glorify their use and our disposition,
Not only in the pre- and post-fabrication.
But also in all concrete education.
Its textbooks are awaiting a revolution,
Calling authors for abandoning tradition,
Creating a basic re-or-i-en-ta-tion,
Combining R.C. and P.C. under one edition,
Laying an entirely new foundation,
As a fresh approach for the future direction.
R.C., partial and/or full prestressing selection,
Each best fits its just optimum adaptation,
Neither one-way, two-way, three-way limitation,
Nor any pretension or post-tension restriction,
To extend Lin's unlimited application.
Nothing is more urgent than this reformation,
For students' time and energy conservation,
And R.C./P.C. epoch-making unification.
The influence of its early completion
Is beyond all the concerned estimation,
Deserving a joint committee creation,
To speed its preparation and publication.
As a last remark, don't forget gap gradation,
For its better vibrating con-so-li-da-tion,
Much lower shrinkage and creep in composition,
Greater e-las-ti-ci-ty e-va-lu-a-tion,
Higher strength and density due to compaction,
Saving much cement for economization,
As an effective hedge to counter inflation,
The best for prestressed concrete condition.

March 1, 1977.

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†Board Chairman, World Open University, Orange, California. (Dr. Li, as president of Tangshan Engineering College, China, bestowed upon T. Y. Lin the BS degree in civil engineering in 1931.)

Historical Perspective on Prestressed Concrete*

by David P. Billington

Comments by Paul W. Abeles and Author

PAUL W. ABELES†

I salute Professor T. Y. Lin, your "Mr. Prestressed Concrete." My first meeting with him took place at the "Western Conference on Prestressed Concrete" in Los Angeles in November, 1952, at which time we both presented papers. Then again in Europe we met at the First FIP Congress in London in 1953, when we discussed among other matters my experience with partially prestressed precast roof structures.

The kind of partial prestressing discussed at that time corresponded to permitting limited flexural stresses (approximately three-quarters of the modulus of rupture). This concept was also developed independently in Germany and called "beschränkte Vorspannung" (limited prestress) which corresponds to Class 2 structures of the CEB classification.

According to the "First Report on Prestressed Concrete" of the Institution of Structural Engineers, London, September, 1951 (which includes various types of prestressed concrete), the preferable solution (according to my proposal of September, 1950)[50] was a case in which the sections under maximum strain are in compression under permanent load; however, temporary hair cracks were permitted to occur under the transient rare maximum service load as the cracks completely closed when temporary loads were withdrawn. This type of construction was then later included in the British Code CP 115 of 1959.

T. Y. Lin apparently recognized from the beginning the advantages of partial prestressing. For example, in the first edition of his book *Design of Prestressed Concrete Structures* (1955), he included a chapter on partial prestress. He introduced in his practice the load balancing method, which is based on the condition that the effective prestressing force balances either the bending moment due to permanent load or that of the frequently occurring load. In the load balancing method Lin avoided any computation of stresses in the elastic stress range and made no reference to the term "partial prestressing." Thus, although his method of load balancing does in fact involve "partial prestressing," he was able to introduce this design method without having to "fight" building code officials for approval.

The advanced thinking of T. Y. Lin in understanding the practical aspects of prestressing can be best appreciated by the dedication in his book: "To engineers, who rather than blindly following the Codes of Practice seek to apply the laws of nature." Thus, T. Y. Lin together with his firms, have used design methods which fully take into account the advantages of prestressing without limiting the design to permanent compressive stresses as originally demanded by Freyssinet.

In passing, I might refer to the Caracas National Race Track which according to p. 37 in the Sept.-Oct. '76 PCI JOURNAL "survived the 1957 earthquake with absolutely no distress." I remember having been present at a lecture by Professor Lin when he reported on this design and was criticized by a speaker that apparently cracks occurred under maximum service load. However, it is the flexibility of the structure which was beneficial as compared to a rigid, brittle construction.

*PCI JOURNAL, V. 21, No. 5, September-October 1976, pp. 48-71.

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I now wish to discuss some aspects of the papers by Professors Alan H. Mattock and David P. Billington.

First, I appreciate Professor Mattock's special reference to my endeavours regarding the introduction of partial prestressing and pointing out its advantages. I find his paper condensing the various aspects of research excellent, but would like to make an amplification to Fig. 2 of the paper.

There were two similar beams cast and tested in Frankfurt and Dresden, respectively. They showed a difference in failure loads, dependent on the concrete strengths. In one girder the steel fractured as the beam was under-reinforced, whereas in the other girder with the weaker concrete, failure occurred at a lower load owing to crushing of the concrete, without fracture of the steel.

Thus, the importance of the failure load design, independent of the condition that there must be no tensile stress under working load, should have become evident at that time.

Now returning to Professor Billington. I am unfortunately forced to disagree with some of his statements and to add some important names to the title "Historical Perspective on Prestressed Concrete," as I have been studying the historical aspects for a considerable time and am perhaps the oldest living specialist concerned with prestressed concrete.

I fully agreed with his statements on Magnel and Finsterwalder to which I wish to make a few additional remarks. I knew Professor Magnel from 1945 until his sudden and untimely death. We became close friends and he visited me often at Kings Cross, London.

Magnel was the first to make tests on partial prestressing with post-tensioned tendons on my suggestion as early as 1946.[51] As a research worker he recognized early the importance of failure load and of the friction losses of steel. He became interested in the progress of my work at British Railways from 1948 until his death. In 1952 he organized the Prestressed Congress in Ghent which became the forerunner of the first FIP Congress in London in 1953.

Magnel participated with Professor Rüsch and myself in a symposium on prestressed concrete in Zurich in 1953. After his untimely death, Professor Riessauw (his successor), A. L. L. Baker, R. H. Evans and myself dedicated the September issue of the periodical *Concrete and Construction Engineering* to him.

As far as Dr. Ulrich Finsterwalder is concerned, his early idea to use high strength steel and to prestress the tensile members of trusses as early as 1934 should be mentioned. He reported on this in the publications of IABSE in 1936 and their Berlin Congress in the same year. The first prestressed truss bridge, according to his colleague, Professor Dischinger, was built in Aue in 1938.

It should have been restressed after creep and shrinkage according to the ideas of Dr. Dischinger, but this was delayed owing to the war when already extensive cracks had occurred. In addition to his great success with bridges built by the cantilever method (Fig. 10) for which the DYWIDAG system is particularly suitable, and to bridges with cable stays (Fig. 11), also bridges with shallow mushroom decks should be mentioned.[52]

After World War II Finsterwalder developed the DYWIDAG system of using high strength hard steel bars as tendons and reported on it and its applications at the IABSE Congress in Cambridge (1952), Lisbon (1956) and Stockholm (1960). Already in Cambridge (1952) he pointed out that the designs were based on limited prestress.

This concept was introduced in Germany (1953) with the assistance of Professor Rüsch in the German Standard by allowing tensile stresses up to three-quarters of the modulus of rupture (i.e., without visible cracks) although the draft was already published in 1951. Nevertheless, it was necessary to provide special non-tensioned steel bars to take up the tensile stresses, which was not included in the otherwise similar practice by British Railways which commenced in 1948.

As Professor Billington points out, the Mangfall Bridge is especially interesting, since it replaced a steel bridge built in 1936. The tender was won by Dyckerhoff and Widmann in stiff competition with steel bridges, as Finsterwalder pointed out at the Stockholm Congress IABSE

(1960). The Mangfall Bridge was built between 1956 and 1960, not 1945.

Now turning to Eugene Freyssinet. His great contribution was the general introduction of prestressed concrete which must be fully recognized, since he realized early the significance of creep and shrinkage losses and the necessity of using high strength steel. This was, however, used previously for prestressing pipes employing circular wound wire.[53]

As a concrete engineer, Freyssinet became famous by building long span arch bridges, e.g., that at Plougastel (three spans of 168 m) mentioned by Dr. Mattock on page 75 and described in Fig. 1 (adjustment of concrete arches). However, by applying his ideas to beams, he unfortunately limited himself to the aim of obtaining permanent compressive stresses in the structure. This idea of "full" prestressing was based on pressure lines in arches and ignored failure conditions.

The first beam tests were carried out in Germany in 1936 (Figs. 2 and 3 of Dr. Mattock's paper) and proved that the loading capacity is affected by ultimate load conditions. Whereas other research workers recognized this, he was so much obsessed by his basic idea of permanent compressive stresses, originating from the arch, that he still in 1950 in a lecture at the Institution of Civil Engineers in London,[54] stated that for prestressed concrete, different safety considerations apply from other materials; furthermore, he made his well-known statement that a half-way house between "fully" prestressed and reinforced concrete is worse than either of the two structural materials.[55]

It was most unfortunate that Freyssinet used at the beginning of the practical application of prestressing his idea to fabricate poles (see Fig. 3 of Billington's paper). Poles must be resilient and capable of taking up occasional excess loads (sometimes even impact loads) without collapsing.

Thus, fully prestressed poles being rather brittle and uneconomical, are not very suitable for this purpose, and Freyssinet wasted valuable time in the introduction of prestressing in 1933. I had experiences in Austria in 1933-38 with the rather flexible spun concrete masts[56] which were quite suitable for the high stress requirements. In fact, R. Bührer designed partially prestressed poles for the Federal Railways in Germany after World War II.[57]

Freyssinet was overly enthusiastic regarding the limitation of his ideas to "full" prestressing, and he convinced his licensees that this was the only solution. For example, T. J. Gueritte in a discussion in 1941 to my suggestion of moderate partial prestress of 750 psi (5 N/mm^2) stated that heavy cracking would occur which would do away with all the advantages of prestressing.[58] The stress in question corresponded to three-quarters of the modulus of rupture.

At the IABSE Congress at Liege in 1948, I was the only one to disagree with a definition of prestressing drafted by a committee including Freyssinet (but in the absence of Magnel) which said that it means artificially counteracting the loading so that permanent compressive stresses develop. This was then later amended to a more general wording by Magnel and Lardy.

At the IABSE Congress in Cambridge in 1952, Finsterwalder, when reporting on his bridges, stated that they were obviously with limited prestress; at the FIP-PCI New York Congress in 1974, as well as at the ACI Symposium in Philadelphia in 1976, Dr. Leonhardt called "full" prestressing a wrong philosophy of Freyssinet.

Be that as it may, I fully agree with Professor Bresler's statement "that even the great innovators cannot always foresee the implication of their inventions."

Professor Billington has completely ignored the work of **Eduardo Torroja** who was, in my view, the greatest structural engineer of this century. He combined a mastery of design with a knowledge of materials and their behavior. As early as 1925, when Torroja was only 26 years old, he designed and built the first effectively prestressed suspended concrete bridge, the Tempul Aqueduct in Spain (Fig. A).

It was not possible to use the initial reinforced concrete design with three spans over the river because the controlling authorities believed that the two intermediate piers might be undermined by the

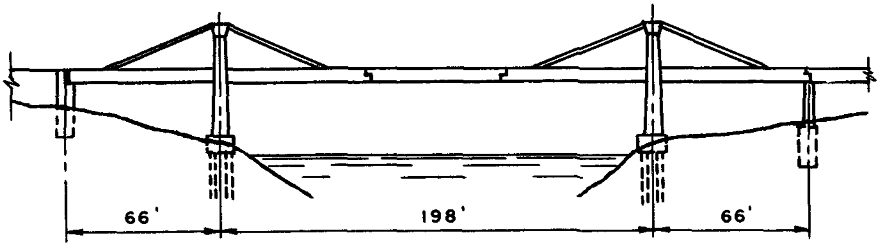


Fig. A. Torroja's prestressed suspended concrete bridge, Tempul Aqueduct.

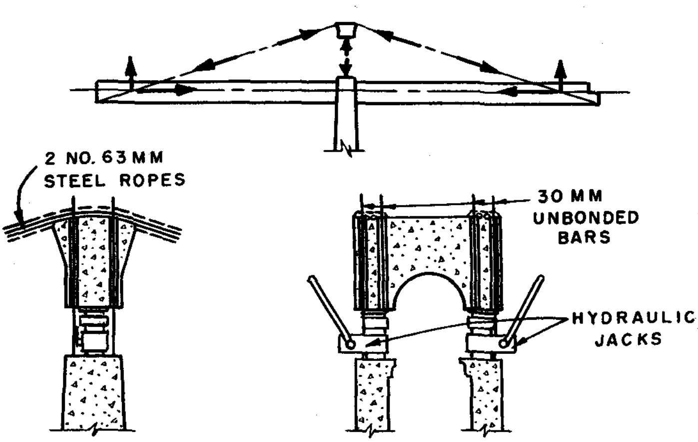
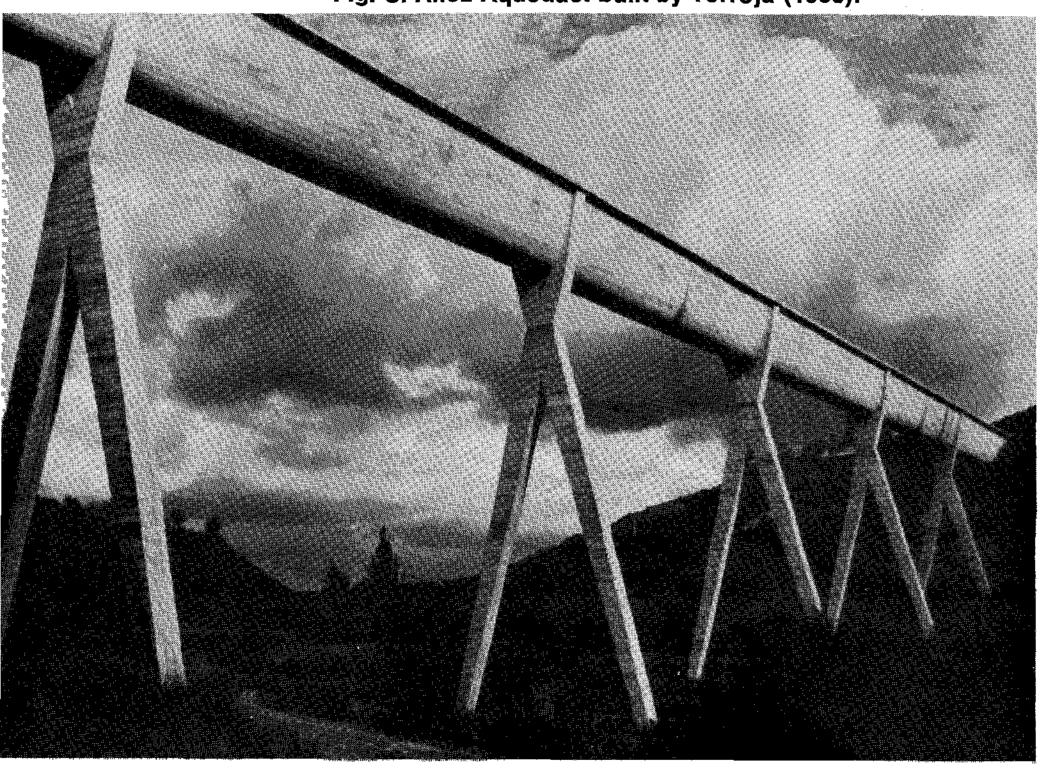


Fig. B. Stresses in tie members and prestressing device used for Tempul.

Fig. C. Alloz Aqueduct built by Torroja (1939).



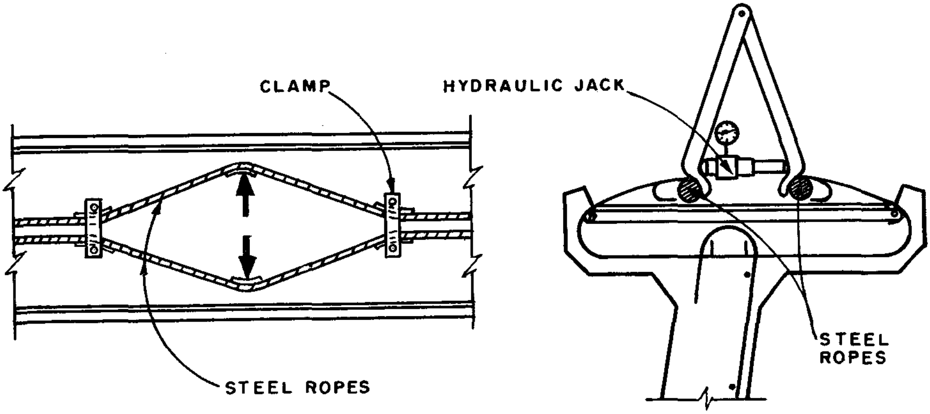


Fig. D. Cable system and jack tightening device used at Alloz Aqueduct.

water. Since there was very little time to prepare a new design for a bridge three times as long, the young Torroja found the ingenious solution of keeping the beams as they were, omitting two intermediate piers and strengthening the two adjacent piers, the two inside spans being supported by prestressed tie members. Torroja provided high strength cables, and the simple method of prestressing by jacks is illustrated in Fig. B.[59]

Another noteworthy example is shown in Fig. C, the Alloz Aqueduct, built by Torroja in 1939. Longitudinal steel ropes were provided at the upper extremities of each channel section and anchored in the concrete. After hardening, each pair of cables was tensioned by forcing them apart as indicated in Fig. D. The two cables were clamped and certain distances forced apart until the required tension was reached. After some time, the jacked levers were again applied to readjust the forces to eliminate the shrinkage and creep losses.[59]

The Italian engineer **Ricardo Morandi,** is also famous for his work in prestressed concrete. As an example, the well-known Maracaibo Bridge in Venezuela is illustrated in Fig. E. Originally a free span of

Fig. E. Morandi's Maracaibo Bridge in Venezuela.

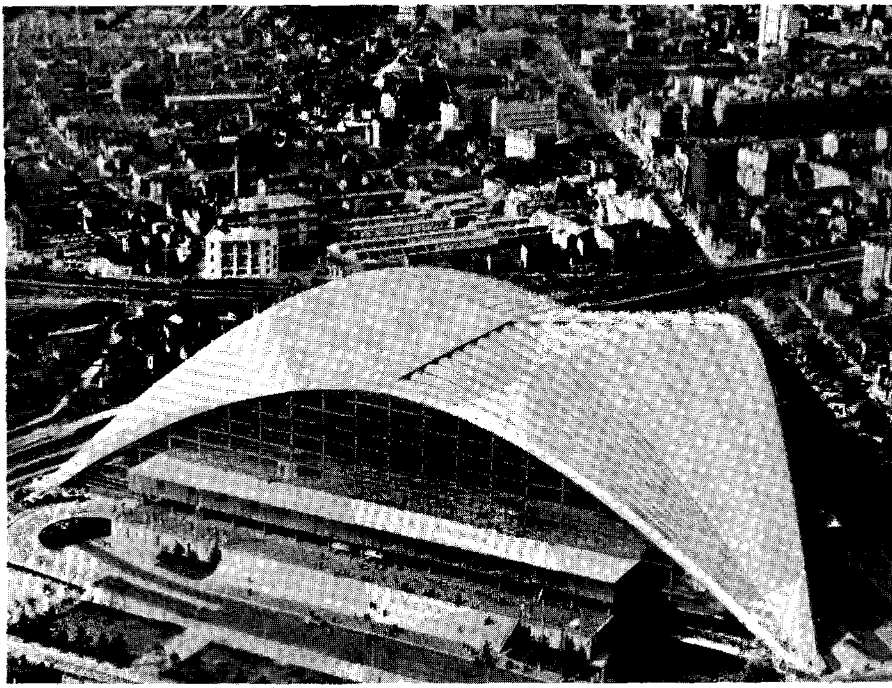


Fig. F. Esquillan's celebrated Exposition Palace in Paris.

1312 ft was incorporated in the design of the six-mile bridge, but for reasons of economy this was reduced. The design includes five spans of 771 ft and a number of shorter spans.

The French designer, **Nicolas Esquillan,** is celebrated for his well known Exposition Palace in Paris (see Fig. F), one of the longest thin shell structures ever built.

And finally, Dr. **Fritz Leonhardt** is famous for his many imaginative bridge designs and bold television towers.

Thus, in my view, these four specialists, viz., Torroja, Morandi, Esquillan, and Leonhardt, and perhaps some others, should be added to the illustrious list of Professor Billington's early pioneers in any "Historical Perspective on Prestressed Concrete."

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AUTHOR'S CLOSURE

I greatly appreciate the remarks of Dr. Paul Abeles on my paper "Historical Perspective on Prestressed Concrete." As is usually the case, Dr. Abeles is correct in what he says about the historical developments. His disagreement with me appears to have two parts: (1) that I have left out the names of many early pioneers; and (2) that I have overestimated Freyssinet's importance. Let me discuss each separately.

My major objective was to identify the primary events in the origin of prestressing as seen from an American perspective, and as they related directly to its introduction in the United States up until 1957. It in no way detracts from the more general historical significance of leading structural engineers like Torroja, Morandi, Esquillan, and Leonhardt to state that they did not play the same central role for America in the early 1950's as did Freyssinet and Magnel.

As for the emphasis on Freyssinet, I must confess that his own account of his past experiences fascinated me and he seemed to me to be at once human, fallible and stirring. But my historical perspective is by no means historical scholarship. I have some idea about the latter, having just completed a book on Robert Maillart, whose own accounts I have correlated carefully with contemporaneous documentation.

As far as I know, no one has done this for Freyssinet, and hence his own accounts must be taken cautiously. That is why I quoted him at length, to give the reader the flavor of the man more than any facts about his background.

Still, from what I can tell now, Freyssinet seems to me to be, along with Maillart, one of the two greatest structural engineers in concrete of the first half of the twentieth century. Now any statement like that is in the same category as identifying LeCorbusier and Wright or Picasso and Klee as the greatest architects and painters of the same period. Such personal opinions usually result in endless arguments.

To be somewhat more precise and to contrast Torroja with Freyssinet, there are three ways in which we might try to describe the greatness of a structural engineer:

First, by his grasp of the scientific basis for structure: the physical behavior of materials and the mathematical prediction of response to load. The test for this grasp lies primarily in the long-term performance of the structures themselves.

The second way of evaluation has to do with the cost of structures and to what extent the designer's works were economical.

Finally, the third test of a designer's greatness lies in his ideas both as expressed verbally in writings and visually in completed structures.

Unfortunately there is so little scholarship available on Torroja, for example, that it simply is not possible to make convincing judgments on his work beyond marvelling at the clearly remarkable visual forms he created in Spain. While I would certainly agree that Torroja is one of the greatest structural engineers of this century, I do not believe that he had any major influence on the rise of prestressing. His early works, referred to by Dr. Abeles, represent fine solutions to specific problems rather than prototype ideas for general use.

Freyssinet appears always to have seen prestressing as a general idea, or as he said himself, "a directing idea." This made him both a kind of stimulating prophet and a rather inflexible proponent of one form of prestressing. Freyssinet's inflexibility, especially with regard to partial prestressing, Dr. Abeles has well described, and in fact has himself made the major contribution to correcting.

This contrast between prestressing, seen either as a radically new idea

(Freyssinet), or as an extension of old ideas would form the basis for a fascinating study in itself, because it represents a specific illustration of the general contrast between cultural ideas of radical reform versus gradual evolution.

Proposals for radical reform, such as those exemplified by the prestressing controversy, always involve two opposing viewpoints—one focusing on the mysterious new powers that will remove previous defects (either crackless concrete in engineering or classless societies in politics) and the other centering on the rational developments that will incrementally improve existing defects (either reduced materials by partial prestressing or increased efficiency by reduced governmental bureaucracy).

The former engages more our emotions while the latter more our rationality.

My own recollection of the situation in American practice of the early 1950's is that Freyssinet's idea of a new crackless concrete helped both to get structures built in the United States and to get research begun in spite of his misunderstanding of the power of partial prestressing.

Finally, I hope that Dr. Abeles will write his own perspective on the history of prestressing. It would be of paramount value to all of us now, and to future students of structural engineering.

Prestressed Concrete For Buildings*

by Alfred A. Yee

Comments by K. Rajagopalan and Author

K. RAJAGOPALAN†

The author must be commended on his paper which presents the economic viability of prestressed concrete in building systems. The writer endorses the cardinal point emphasized in the paper, namely, that efforts must be directed towards achieving savings in materials which really means savings in energy.

The author has detailed the advantages of several prestressed concrete units such as solid, voided and stemmed units and indicated their economic spans. The writer, however, believes that these spans must be arrived at using optimization principles.[5,6]

A methodology of optimization of prestressed concrete solid and voided slabs and the associated computer experiments have been described by the writer elsewhere.[7] However, the following discussion (which summarizes the major results of Ref. 7) is offered in the hope that it will add value to the author's paper.

Table A shows a comparison of the optimal results for two prestressed concrete slabs, one solid and the other voided with two circular voids of 4 in. diameter. The AASHTO stress regulations for pretensioning are used and a 20 percent loss in prestress is considered. The slabs are 12 in. wide. The thick-

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Table A. Minimum cost designs (in dollars per sq ft) of solid and voided slabs (AASHTO Code). Live load = 75 psf; f'_c = 6000 psi.

Span (ft)	Solid Plank					Voided Plank (2 of 4 in.)					Percentage Savings in Minimum Cost Design of Voided Slab			
	R	h (in.)	F_i (lbs)	Cost U_s	Weight (lb/ft)	R	h (in.)	F_i (lbs)	Cost U_s	Weight (lb/ft)	Concrete	High Tensile Steel	Cost	Weight
30	30	11.10	53559	2.00	140	30	9.30	50131	1.58	91	35.08	6.40	21.00	35
	40	9.84	60948	2.32	124	40	8.26	56224	1.78	78	37.33	7.75	23.28	37
	50	9.03	67091	2.61	114	50	7.60	61239	1.96	70	39.02	8.72	24.90	39
	60	8.45	72453	2.88	107	60	7.13	65573	2.13	64	40.39	9.50	26.04	40
	70	8.02	77267	3.14	102	70	6.78	69429	2.28	60	41.56	10.14	27.39	41
	80	7.67	81672	3.38	97									
NW	145	6.42	104402	4.76	82	78	6.55	72378	2.39	57				
35	30	12.49	69626	2.42	157	30	10.43	65509	1.96	105	33.25	5.91	19.01	33
	40	11.01	78279	2.78	139	40	9.20	72687	2.20	90	35.45	7.14	20.86	35
	50	10.06	85473	3.11	127	50	8.42	78609	2.41	81	37.11	8.03	22.51	37
	60	9.39	91752	3.41	119	60	7.87	83739	2.59	74	38.48	8.73	24.05	38
	70	8.87	97389	3.69	113									
	80	8.47	102548	3.96	108									
NW	121	7.40	120326	4.95	95	69	7.50	87828	2.75	69				
40	30	13.87	87438	2.87	175	30	11.57	82623	2.38	120	31.68	5.51	17.07	31
	40	12.18	97350	3.27	154	40	10.16	90879	2.65	102	33.77	6.65	18.96	33
	50	11.09	105592	3.63	141	50	9.26	97704	2.88	91	35.38	7.47	20.66	35
	60	10.32	112785	3.96	131	60	8.62	103624	3.09	83	36.76	8.12	21.97	36
	70	9.73	119243	4.27	124									
	80	9.26	125153	4.56	118									
NW	104	8.46	137562	5.21	108	62	8.53	104588	3.13	82				

ness of the plank is denoted by h and F_i represents the initial prestressing force.

The table shows that minimum cost designs are possible over a wide range of R, the ratio of the cost of concrete (U_c), in dollars per cubic yard, to the cost of high tensile steel strands (U_s), in dollars per pound. Each horizontal entry in the table represents the minimum cost design for a particular value of R except lines MW which indicate minimum weight designs.

It can be seen that the minimum cost designs are bounded by the minimum weight design and this occurs at a limiting value of R. This means that when the costs of concrete (including the cost of placement) and of steel strands (including the cost of prestressing) in a particular locality are such that their ratio is greater than or equal to this limiting value, no minimum cost design is possible (minimum cost design coincides with the minimum weight design).

The limiting value of R varies over a wide range being dependent on span, live load, concrete strength, prestress losses, type of slab etc. In particular, Table A shows that this limiting value of R for a solid plank to be 145, 121 and 104, respectively, for spans of 30, 35, and 40 ft. For the hollow-core slab the limiting values of R are 78, 69 and 62.

Suppose, for example, in a certain county (or country) the value of R is 40. This means that minimum cost designs are possible for all spans considered in Table A both for the solid and the hollow-core planks. On the other hand, in a different locality with an R value of 80, minimum cost designs over these spans are possible only for solid slabs and all the hollow-core planks can only be minimum weight planks.

It is interesting to note from Table A that in a place where R is 60, adoption of a minimum cost hollow-core slab in place of a minimum cost solid slab for a 30-ft. span would result in 40 percent savings in concrete and a 9.5 percent savings in high tensile steel.

The corresponding savings in cost and weight are 26 and 40 percent, respectively. The savings are thus considerable. Much more savings in high tensile steel is realizable by any firm if a minimum cost hollow-core slab is substituted for a minimum weight solid slab, a design which is normally adopted. The savings, however, decrease with increasing spans.

Table B presents the possible savings at two different cost ratios of 40 and 60 for a typical span of 30 ft, the losses in prestress being assumed 20 percent as before. Three

Table B. Percentage of savings for 30-ft span.

Live Load (psf)	f'_c (psi)	Voided Plank (2 of 4 in.) Limiting R	R	Concrete	High Tensile Steel	Cost	Weight	Voided Plank (1.5 of 6 in.) Limiting R	R	Concrete	High Tensile Steel	Cost	Weight
75	5000	61	40	37.44	7.95	23.08	37	45	40	57.74	18.04	38.46	57
			60	40.44	9.63	26.21	40		60	-	-	-	-
	6000	78	40	37.33	7.75	23.28	37	59	40	57.56	17.72	38.36	57
			60	40.39	9.50	26.04	40		60	-	-	-	-
	7000	97	40	37.22	7.56	23.38	37	75	40	57.29	17.42	38.53	57
			60	40.30	9.37	26.48	40		60	59.44	21.87	42.51	59
125	5000	65	40	31.31	5.30	18.87	31	48	40	50.75	11.17	31.70	50
			60	34.66	6.40	21.65	34		60	-	-	-	-
	6000	82	40	31.20	5.12	19.01	31	61	40	50.48	10.87	31.94	50
			60	34.59	6.28	21.71	34		60	54.12	14.16	35.78	54
	7000	100	40	31.09	4.95	18.77	31	75	40	50.66	9.12	31.42	50
			60	34.51	6.16	21.85	34		60	54.21	12.95	35.38	54

different concrete strengths, two live loads and two types of hollow-core slabs have been considered.

The comparison here is between two voided slabs, one of 12 in. width containing two voids of 4 in. diameter, and another of 24 in. width with three voids of 6 in. diameter. These dimensions ensure a uniform minimum concrete of 2 in. between the voids and 1 in. at the ends in both the slabs.

It should be noted that the writer's results satisfy AASHTO stress constraints only. Checks for ultimate moment must of course be carried out. These details and other considerations such as the inclusion of erection cost in the optimum design appear elsewhere.[7]

It would be interesting to see how the optimal results presented in Table A would compare if the provisions of ACI 318-71 are to be

Table C. Minimum cost designs in dollars per sq ft of solid and voided slabs (ACI 318-71). Live load = 75 psf; f'_c = 6000 psi.

Span (ft)	Solid Plank R	h (in.)	F_i (lbs)	$\frac{\text{Cost}}{U_s}$	Weight (lb/ft)	Voided Plank (2 of 4 in.) R	h (in.)	F_i (lbs)	$\frac{\text{Cost}}{U_s}$	Weight (lb/ft)	Percentage Savings in Minimum Cost Design of Voided Slab: Concrete	High Tensile Steel	Cost	Weight
30	30	11.96	35631	1.76	150.18	30	10.10	35155	1.38	100.67	33.05	1.34	21.59	32.97
	40	10.30	45347	2.10	129.60	40	8.71	43272	1.60	83.48	35.76	4.58	23.81	35.59
	50	9.32	52802	2.40	117.42	50	7.89	49450	1.79	73.39	37.80	6.35	25.42	37.50
	60	8.65	59034	2.68	109.16	60	7.34	54557	1.96	66.61	39.34	7.58	26.87	38.98
	70	8.15	64465	2.93	103.08	70	6.94	58974	2.12	61.66	40.53	8.52	27.65	40.18
	80	7.77	69334	3.18	98.37	80	6.63	62900	2.26	57.87	41.61	9.28	28.93	41.17
NW	175	6.11	102412	5.15	78.20	98	6.22	69138	2.51	52.79				
35	30	13.54	49592	2.16	170.11	30	11.37	48783	1.75	116.85	31.49	1.63	18.98	31.31
	40	11.58	61014	2.54	145.92	40	9.74	58343	2.00	96.60	33.97	4.38	21.26	33.80
	50	10.43	69786	2.88	131.60	50	8.77	65634	2.22	84.69	35.99	5.95	22.92	35.65
	60	9.64	77104	3.19	121.88	60	8.12	71675	2.42	76.67	37.48	7.04	24.14	37.09
	70	9.06	83489	3.48	114.74	70	7.65	76908	2.60	70.81	38.67	7.88	25.29	38.29
	80	8.61	89214	3.75	109.20	80	7.28	81568	2.76	66.31	39.76	8.57	26.40	39.28
NW	145	7.03	117898	5.30	90.02	86	7.11	84030	2.85	64.16				
40	30	15.10	65294	2.59	189.95	30	12.65	64139	2.14	133.14	30.09	1.77	17.37	29.91
	40	12.86	78413	3.01	162.17	40	10.77	75135	2.44	109.85	32.53	4.18	18.94	32.26
	50	11.53	88487	3.39	145.73	50	9.66	83534	2.69	96.13	34.37	5.60	20.65	34.04
	60	10.62	96892	3.73	134.56	60	8.91	90504	2.91	86.88	35.81	6.59	21.98	35.43
	70	9.96	104225	4.05	126.36	70	8.36	96551	3.11	80.11	37.08	7.36	23.21	36.60
	80	9.44	110800	4.35	120.00									
NW	125	8.03	134631	5.54	102.80	77	8.07	100197	3.24	76.51				

satisfied. This is done in Table C.

A comparison with the costs presented in Table 6 of the author's paper perhaps underlines the need to consider optimization principles in prestressed concrete design.

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AUTHOR'S CLOSURE

The author appreciates Dr. Rajagopalan's comments and contributions with regard to optimization principles as they may be applied to the author's paper.

The author is favorably impressed with the discussor's concept for cost determination of various concrete structural framing units and commends him for his in-depth approach to the analysis of various factors and elements that make up costs.

However, it must be stated that the ultimate determination for economy in various structural units will have to include practical considerations of manufacture, storage, transportation, erection and structural accommodation of building utilities, architectural and functional requirements.