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

Band: 12 (1984)

Artikel: Progress and problems: today and tomorrow

Autor: Lin, T.Y.

DOI: https://doi.org/10.5169/seals-12139

Progress and Problems — Today and Tomorrow

Progrès et problèmes — Aujourd'hui et demain

Fortschritt und Probleme — heute und morgen

T.Y. LIN
Board Chairman
T.Y. Lin International
San Francisco, CA, USA

T.Y. Lin served as Chief Bridge Engineer and Chief Design Engineer for several Chinese Railways, 1933–46. Allowing that, he taught for thirty years at University of California, Berkeley, including service as Chairman of Structural Engineering and Laboratory. He has contributed 100 papers and three books to the profession and has won numerous awards.

SUMMARY
Progress and problems in structural engineering are classified in three broad categories: Design, Analysis and Construction. Examples are presented to illustrate how progress has created problems which will lead to further progress and new problems. Internal prestressing develops into external prestressing. Prestressed concrete paves the way for cable stressed steel. Sudden and catastrophic failures force engineers into the concept of redundancy. Today's emphasis on global stress criteria and analyses will have to be shifted toward local detailing and modeling. Design-construction packages and value-engineering will require close collaboration between engineers and constructors.

RESUME
Les progrès et problèmes dans les structures sont classés dans trois grandes catégories: projet, analyse et construction. Des exemples sont présentés pour illustrer la façon dont le progrès a engendré des problèmes qui vont conduire à d'autres progrès et de nouveaux problèmes. La précontrainte se développe vers une précontrainte extérieure. Elle montre également le chemin à l'acier précontraint. Des ruptures soudaines et catastrophiques forcent les ingénieurs à prévoir un surnombre d'éléments. L'accent actuel qui porte sur des critères et analyses globaux devra être déplacé vers des soins constructifs et de modélisation. Une collaboration étroite entre projet et exécution sera indispensable.

ZUSAMMENFASSUNG
Der Fortschritt und die Probleme im konstruktiven Ingenieurbau können in drei Kategorien eingeteilt werden: Entwurf, Bemessung und Ausführung. Es werden Beispiele aufgeführt, um zu zeigen, wie der Fortschritt neue Probleme aufwarf, welche zu weiterem Fortschritt und zu neuen Problemen führen werden. Die innere Vorspannung entwickelt sich zur äusseren Vorspannung. Vorgespannter Beton ebnet den Weg für vorgespannte Stahlbauten. Plötzliche und katastrophale Einstürze zwingen die Ingenieure zu Entwurfskonzepten mit Redundanz. Die heutige Betonung der globalen Spannungskriterien und der Bemessung wird sich verlagern müssen auf das Detail und die lokale Modellierung. Entwurf und Ausführung wird eine enge Zusammenarbeit zwischen Ingenieur und Unternehmer fordern.

Progress and problems persist in structural engineering as they do in other fields of endeavor. Many of these have been brought up and eloquently expounded upon in previous papers presented at this conference. The purpose of this paper is not to summarize what has been said, but rather to pinpoint certain areas which perhaps have not been sufficiently included and to illustrate them with examples familiar to the author. These examples will be shown by slides during the oral presentation and some will be included here.

It may be proper to discuss progress and problems under three headings: design, analysis and construction. Progress usually results when the introduction of new materials, new equipment and new structural theory leads to new methods of design, analysis and construction. The very nature of progress itself requires the exercise of control. Lack of control could result in problems both today and tomorrow. Hence, the key word, control, will be frequently mentioned in this paper.

1. DESIGN CONTROL

Recent progress in structural design has centered around prestressing, which enables the control of stresses and strains in high-strength steel and concrete. Engineers of today have the ability and tools to fully utilize the high strength of these materials to control structural behavior. In addition, such control has been extended to external prestressing, which was initiated by Freyssinet but is only now being fully explored.

1.1 External prestressing can be produced by jacking a structure against its supporting foundations or by moving one part of the structure relative to the other, using jacks, tendons or other devices. Some examples are shown below:

1.1.1 An 18-story apartment building in South San Francisco, 60 ft. wide by 330 ft. long (18 m x 100 m), was built of partition walls and flat slabs, with no beams or columns for the entire structure, Fig. 1A. In order to economize construction, the floors were 5 in. (16 cm) flat slabs, post-tensioned along the length of the building and supported by the walls at 15 ft. (4.5 m) centers. Thus 330 ft. (100 m) long continuous slab tendons run from one end of the building to the other. For architectural and planning reasons two elevators are needed, one at each end of the building. It was necessary to control the shortening of the long slabs by prestressing which could pull the elevators inward relative to the foundation. To accommodate this movement, the walls around the elevators were cut loose from the foundation and supported on rollers made of re-bars lubricated with graphite. Fig. 1B. Since the 18 floors of flat slabs were post-tensioned with a total force of 11,000 tons (10,000 m.t.) each elevator moved inward by 1 inch (2.5 cm) during the course of construction, thus relieving much of the elastic and shrinkage strain of the deck slabs versus the foundation. The sliding interfaces around the elevators were eventually welded and grouted to resist earthquakes.

1.1.2 The 23rd Avenue Bridge in Oakland, California was post-tensioned along its entire length and had no expansion joint in the end spans. In order to compensate for the shortening in the deck, jacks were applied at the bottom of the abutments forcing each abutment to move inward over 1 in. (2.5 cm) Fig. 2. The elastic shortening of the top deck was expected to be 1/2 in. (1.2 cm), with 1/2 in. (1.2 cm) of shrinkage and creep to take place in a year or two, and another 1/2 in. (1.2 cm) in the course of time. The initial 1-in. (2.5 cm) movement controlled the relative strain between the top and bottom of the abutment, to within 1/2 in. (1.2 cm) at any time, as versus 1-1/2 in. (3.7 cm) otherwise expected.

1.1.3 The Lewiston-Clarkston Bridge with a center span of 600 ft. (180 m) was constructed by balanced cantilevering, Fig 3A. After the 300 ft. (90 m)cantilever met at mid-span, jacks were inserted therein to push the deck apart by 2 in. (5 cm), Fig. 3B. This additional compression would compensate for the future shortening of the deck under prestress, thus obtaining a more favorable stress distribution in the structure. Additionally, the use of double walls at each main pier helped to increase the flexibility of the structure, thus minimizing the stresses produced by shrinkage and creep.

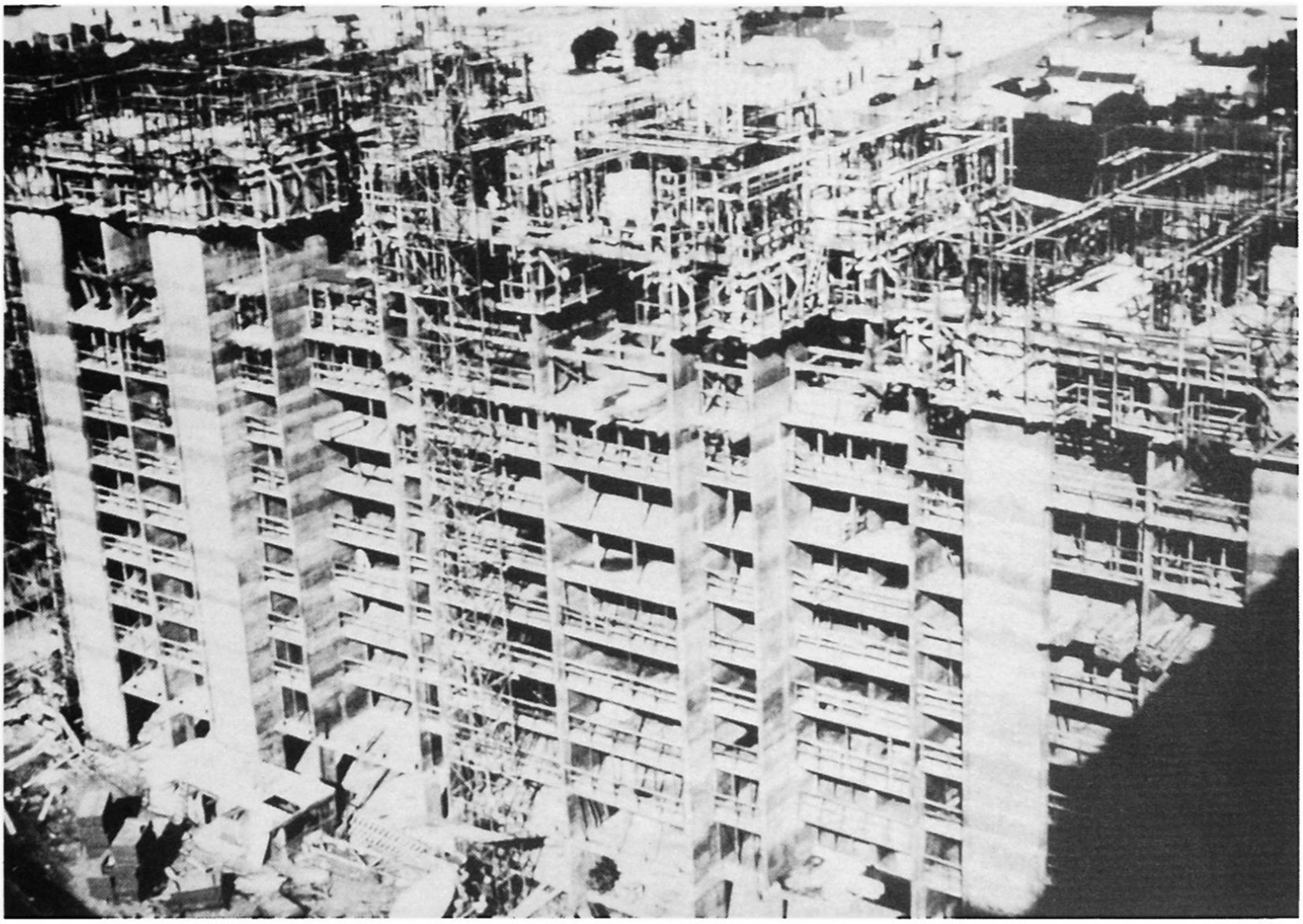
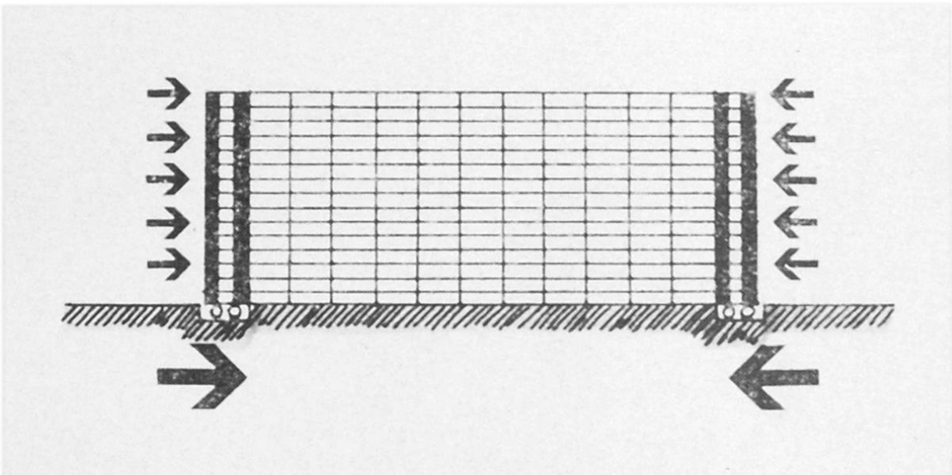


Fig. 1A

Fig. 1B



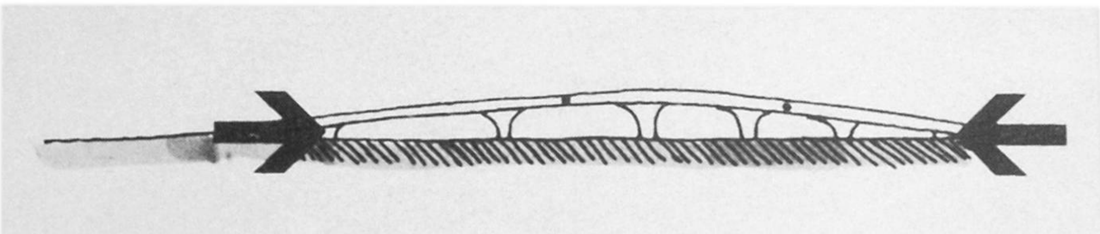


Fig. 2

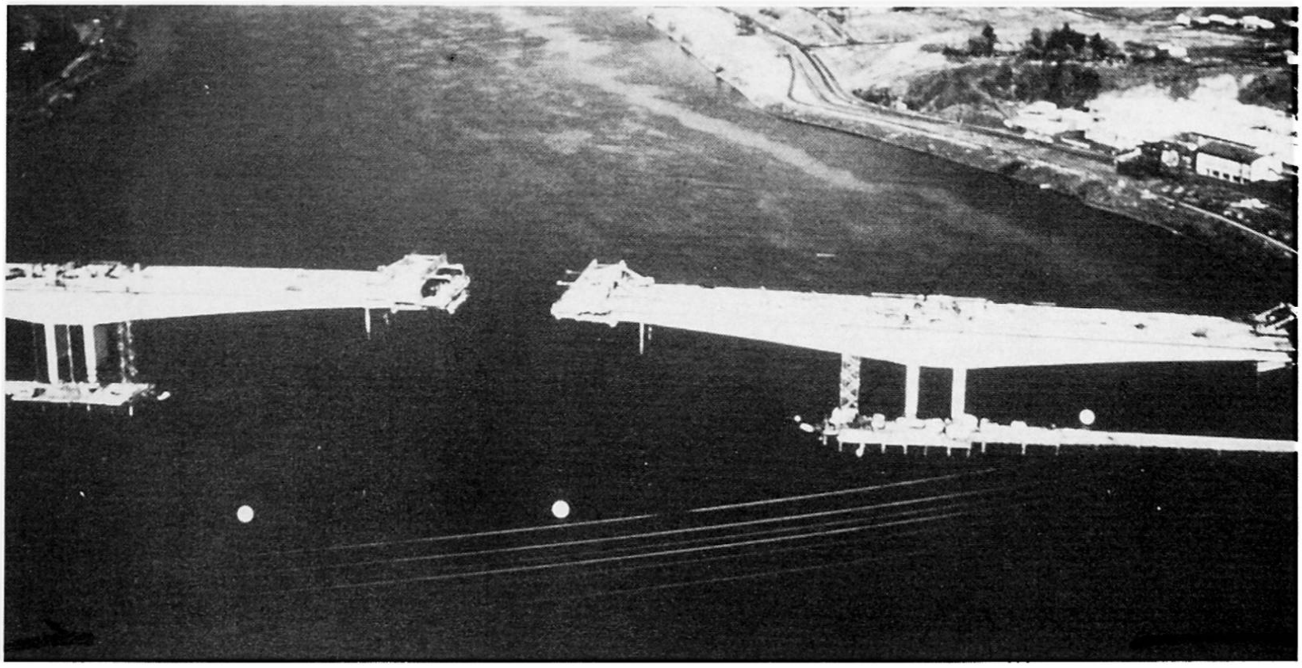


Fig. 3A

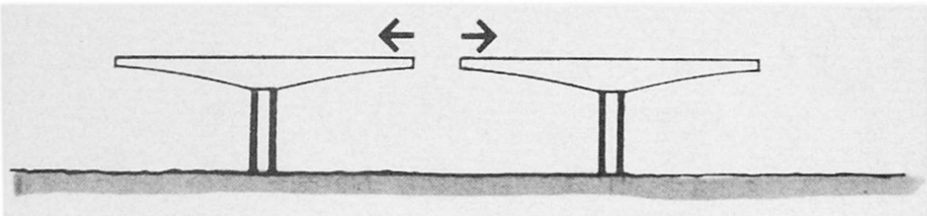


Fig. 3B

Fig. 4A

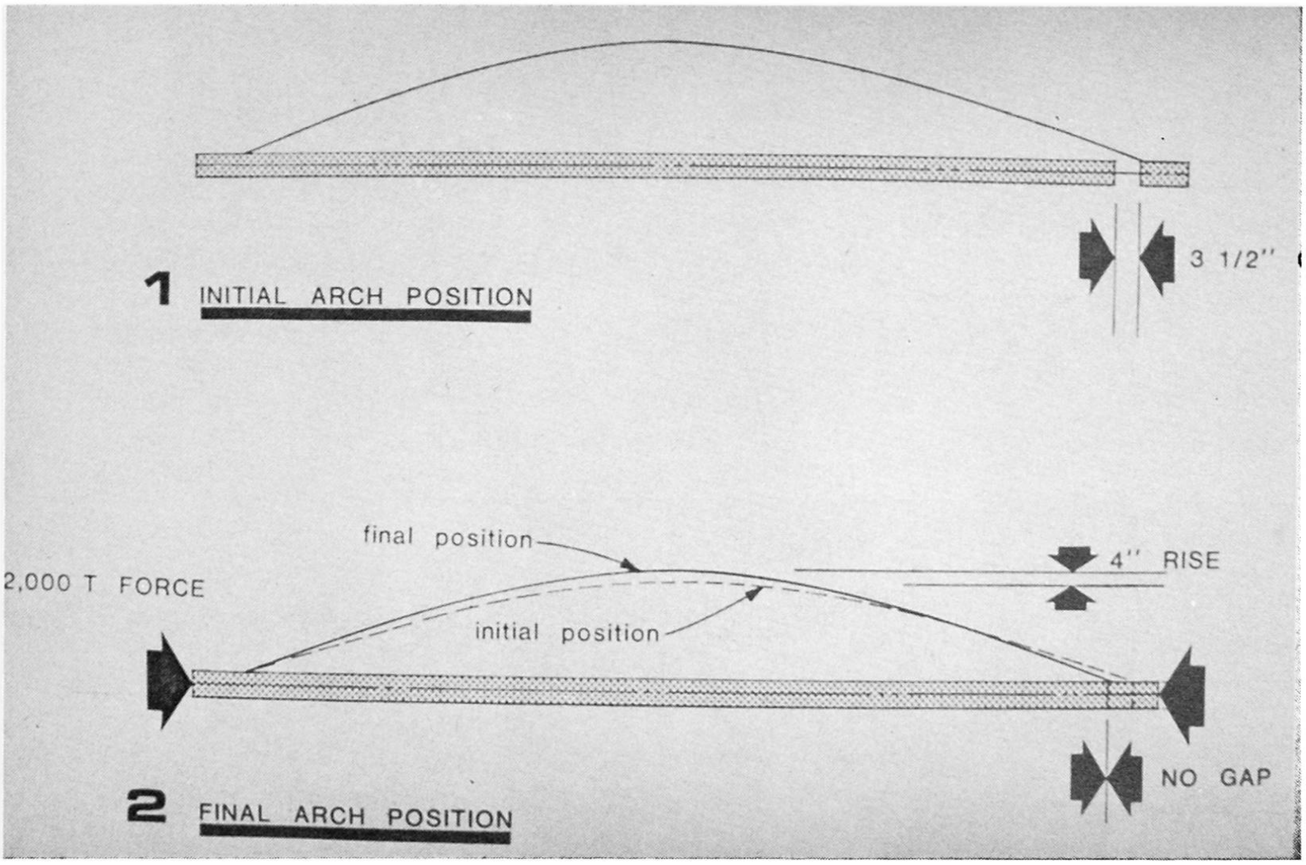

1 INITIAL ARCH POSITION
3 1/2"
final position
2,000 T FORCE
4" RISE
initial position
NO GAP
2 FINAL ARCH POSITION

Fig. 4B

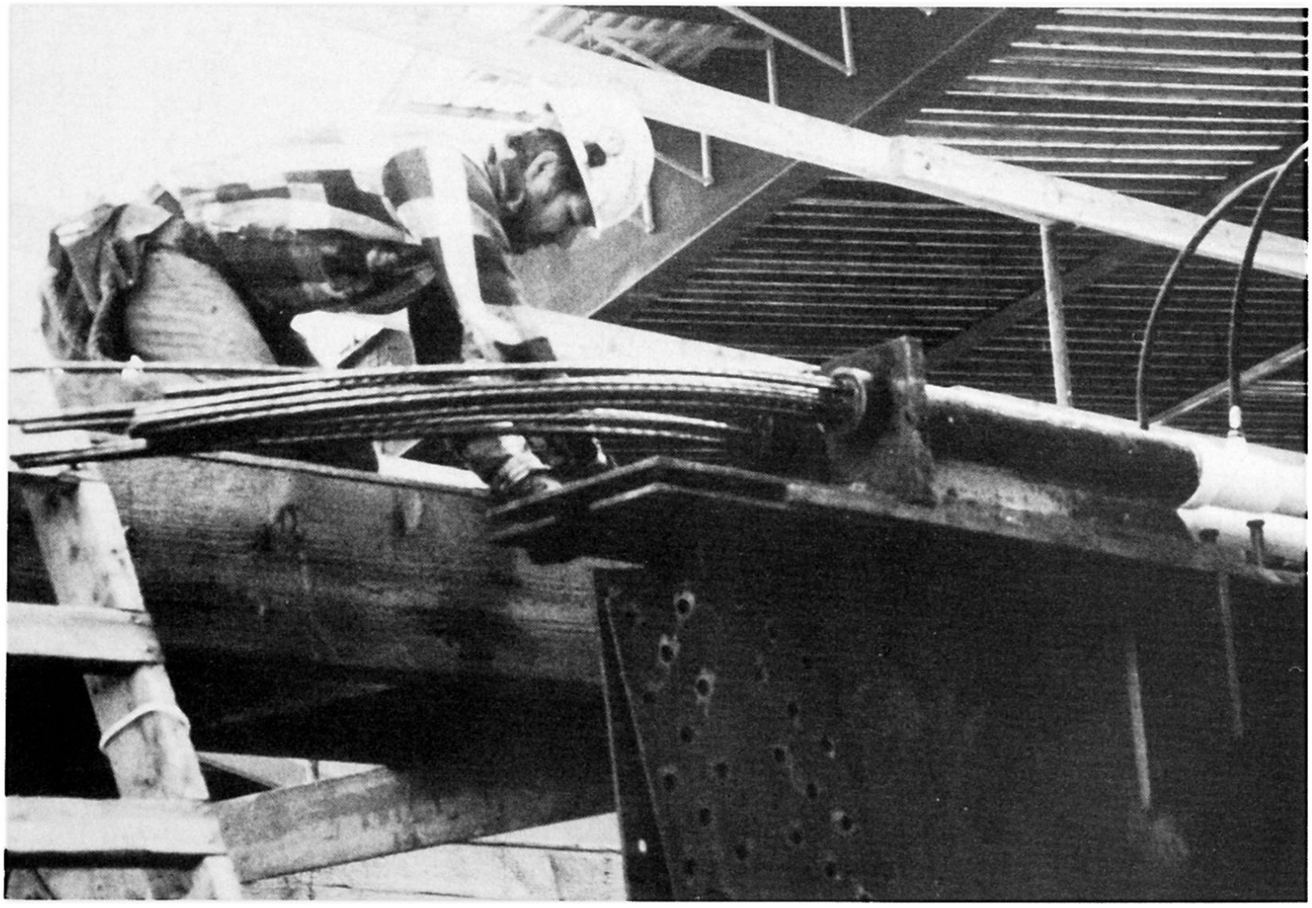


Fig. 5

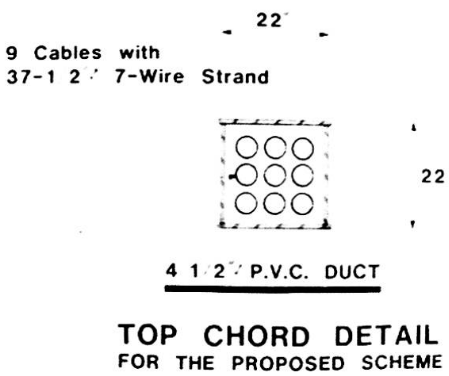


Fig. 6A

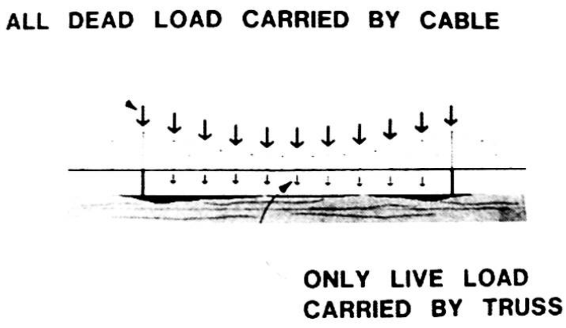


Fig. 6B

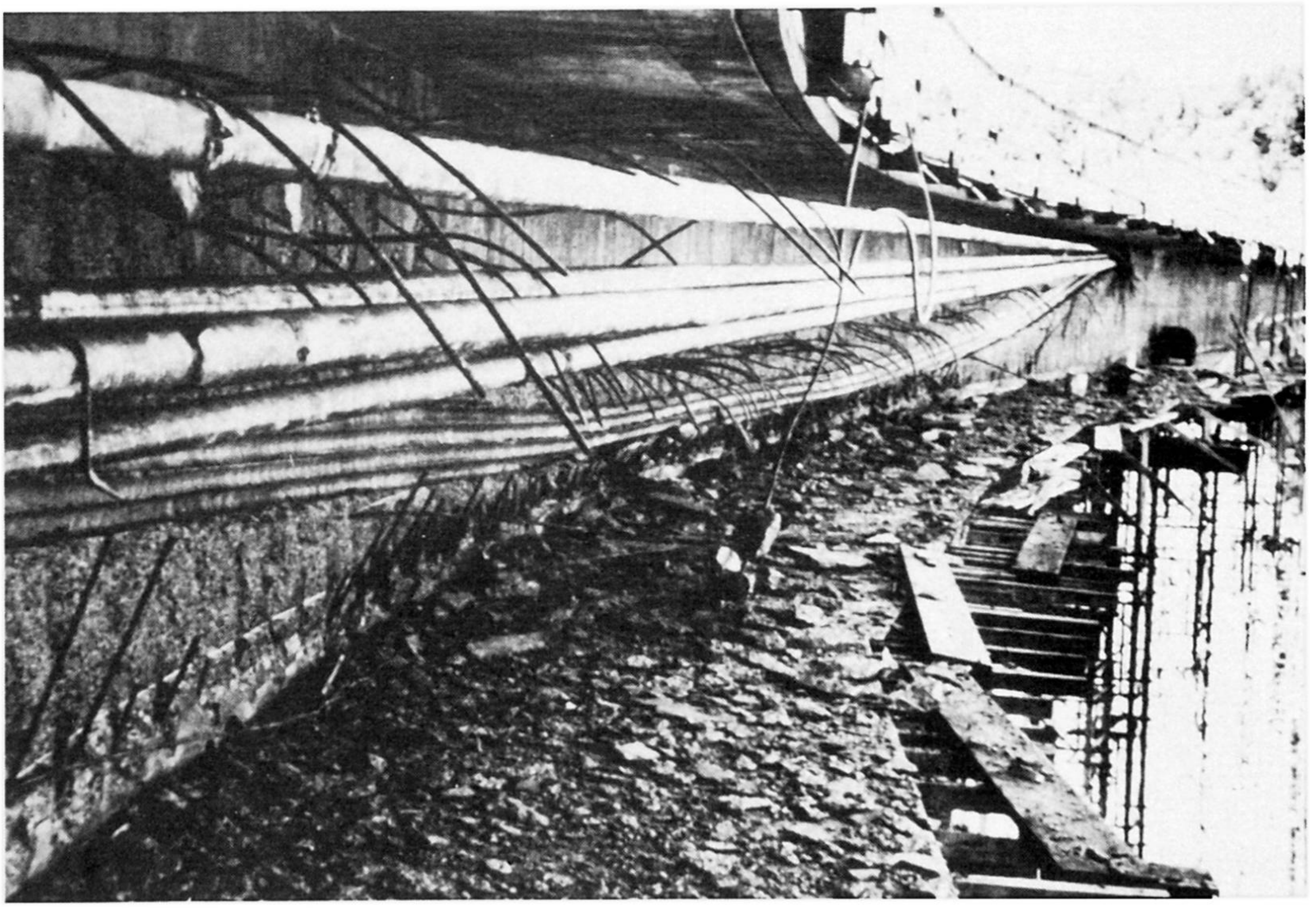


Fig. 7

1.1.4 The Moscone Center has a columnless underground span of 300 X 800 ft. (90 m x 240 m), Fig. 4A. Eight pairs of arches support a heavy roof of concrete covered by 3 ft. (0.9 m) of earth. In order to control stresses in the arches, their ties were post-tensioned to close a 3-1/2 in. (9 cm) gap provided at one end of each arch, Fig 4B. With half of the deadload on the arch, they were stressed to move inward to close the 3-1/2 in. (9 cm) gap causing the arch crown to camber upward by 4 in. (10 cm). This pre-strain controlled the stresses and strains in the arches, providing additional strength to resist the dead and live loads.

1.2 Another progress in the control of stresses is exhibited in the development of cable-stressed steel. Tendons are prestressed against structural steel members, often in composite action with concrete and reinforced concrete. Thus, engineers can better control and fully utilize the strength of cables, structural steel, reinforcing bars, and concrete. This is illustrated by two examples below:

1.2.1 Bonner's Ferry Bridge, Idaho, has post-tensioned cables embedded in the concrete slabs over the piers to resist negative moment. An anchorage detail was devised which connected the cable anchorages to the steel I-girders, Fig. 5. This prestressing produced stresses in the concrete deck as well. The bridge was bid against an alternative design using prestressed concrete and proved to be much more economical.

1.2.2 A study was made to build five spans of a continuous steel truss bridge with span lengths up to 875 ft. (266 m) over the Mississippi River. By imbedding tendons in the top chord of the truss, Fig. 6A, and stressing them, a cable-stressed suspension bridge is obtained with the steel trusses serving merely as stiffening and erection framework. Preliminary estimates indicate a savings of 35% of the superstructure costs because during construction the trusses will only have to carry their own weight which will be transferred to the cables once they are stressed, Fig. 6.A. When finished, the trusses will carry only live load which is a very small percentage of the total load. This example illustrates the desirability of stress control, not only in the final stage but also during construction.

2. ANALYSIS CONTROL

Development of modern computers and software have enabled efficient and accurate stress analysis of complicated structures. Presently, these programs are more often applied to check stress limitations and not so much for achieving behavioral control such as deflection and vibration responses. There has also been an overemphasis on global stress calculations, which tend to divert our attention from a number of local problems. Some examples are described in the following:

2.1. Radial stress from tendons along curves: Failures have occurred in several structures resulting from local stresses produced by the radial force around a sharp curve, particularly where the tendons are bundled, Fig. 7. It is well known that for a statically determinate structure, the global tensile force produced by prestressing coincides with the global compression force produced in the concrete. However, little attention has been paid to the high, local stresses exerted by the tendons on a small area of concrete. Immediately adjacent to the tendons, the concrete cover acts as a thin slab under the action of these radial forces. If not properly reinforced, local failure could occur. Furthermore, the web of a concrete box housing these tendons is subjected to a concentrated lateral load from them, thus producing an overall bending of the web. These stresses are not usually computed, since they do not produce failure until the sharp curvature is combined with other unfavorable conditions. It is time that both our design criteria and our computer capabilities are aimed at these local conditions in addition to global stresses.

2.2 Anchorages and blisters: In post-tensioned structures, high stresses occur near the anchorages, particularly when the load is applied eccentrically such as at blisters. Our design criteria and computer programs are only beginning to be applied to these locations. In addition, one must consider the construction problems involved including congested re-bars and concreting. The interaction between local and global stresses make an exact analysis difficult and time-consuming. These problems must be attacked to attain further progress.

2.3 Dynamics of structures: As we delve into new areas of progress, structures become more slender and more sensitive to vibration. An 18-ft. (5.5 m) wide concrete pedestrian bridge in Oakland, California has spans up to 130-ft. (40 m). This 4-ft. (1.2 m) deep, post-tensioned box girder is supported by 3-ft.-diameter (0.9 m) solid reinforced concrete columns up to 35 ft. (10.6 m) high. When first opened to traffic, the bridge experienced noticeable side-sway when pedestrians surged across after a ball game. The walkway was closed pending remedy. Eventually, 1.5-ft. (0.46 m) thick walls, extending 3.5-ft. (1.06 m) on both sides were added to stiffen the tall columns, Fig. 8. In addition, 15-in. (38 cm) diameter steel pipe-struts were installed to increase vertical stiffness. Extremely sophisticated

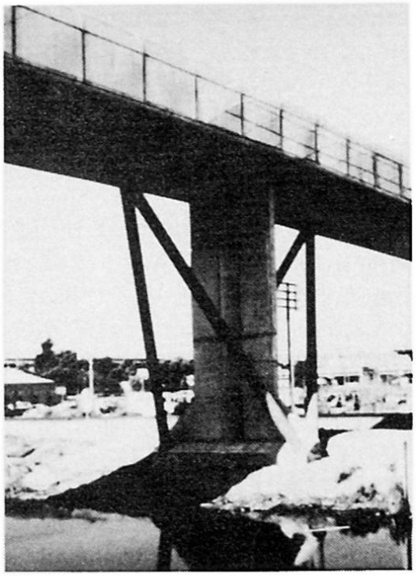


Fig. 8

dynamic analyses were conducted to study the behavior of this bridge, but only a general conclusion was reached. It is believed that the width of the structure coupled with the slenderness of the columns resulted in a type of pinning action of the deck on top of the columns. This indicates that, while we have the available scientific tools for accurate dynamic analysis, we are not ready to arrive at practical recommendations and guides for design.

3. CONSTRUCTION CONTROL

Progress in construction equipment and methods have enabled new sequences and required staging during construction. When a design is based on one type of construction, the Contractor often chooses to employ a different method, such as cast-in-place vs precasting or even a different material, say steel instead of concrete. Design-build packages and value engineering re-designs have emerged in this competitive market. These often require sophisticated construction methods with which contractors are not experienced. In an attempt to arrive at procedures to save time and money, the contractor and his engineers may not realize the difficulties involved and, as a result, problems surface and litigation may result. The following will illustrate one case where versatility is provided in design so the contractors could have control. The second case illustrates a failure resulting from lack of control during construction.

3.1 Case 1: Wing Sections for Bridge Segmental Construction

The wing segmental system of bridge construction, Fig. 9A, was developed in the early 1970's for the Elevated Roadway at the San Francisco International Airport, Fig. 9B. Subsequently, it was applied to six intersections in Bogota, Colombia, where the design-build package won an international competition. Currently, a similar type of construction is proceeding on the Connaught Street Bridge in Vancouver, British Columbia. It has also been chosen by the Texas Department of Highways and Public Transportation as an economical and attractive way to increase the capacity of the I-10/I-35 Y-Interchange in San Antonio.

The versatility of the system lies in its adaptability to various construction methods, combining in-place and precast concrete for either the wings or the spine girders based on the requirements of the project. For example, the spine girder can be of solid or voided concrete, concrete or steel box sections, precast or in-place concrete and others. They can be constructed on falsework or on steel launching girders above or below the spine girder. The wing sections can be precast in one or two pieces, can be integrated with in-place concreting, and can entirely be poured in-place using a launching girder. This means that one design can be made which permits alternate construction methods while maintaining the required shape and geometry of the structure.

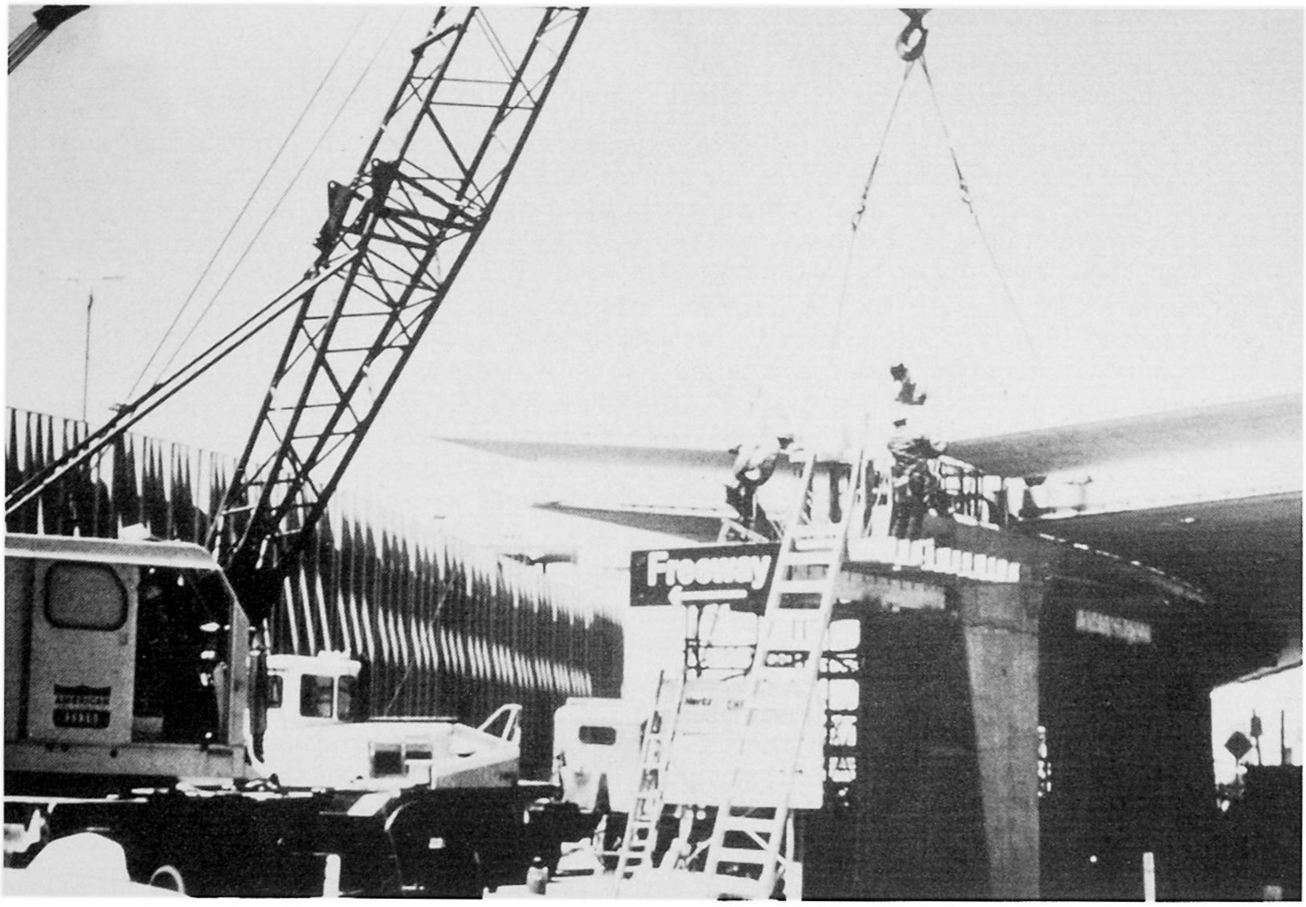


Fig. 9A

Fig. 9B

3.2 Case 2: Failure of a Segmental Construction

A failure occurred in August 1982 during the placement of a precast segment when a hinge collapsed at about the quarter point of the 377-ft. (115 m) span, Fig. 10A.

After extensive study and investigation, it became clear that the failure resulted from a serious underdesign of the hinge which possessed an ultimate factor of safety of only 1.02. For cantilever construction, the bridge was made continuous to carry the precast segments as they were erected toward the center of the span, Fig. 10B. The moment resistance at the hinge was supplied by unbonded top tendons along the top of the box acting against concrete blocks along the bottom. Apparently a computer program was devised to compute stresses in the steel and concrete indicating that the maximum tensile stress in the concrete at the top was zero and at the bottom was 2 ksi (140 kg/cm^2) compression. The unbonded, temporary tendons averaged about 100 ft. long and were stressed to 170 ksi (11900 kg/m^2) during construction. All these stresses conformed with normal limitations. Unfortunately, no ultimate strength calculations were made, and therefore, the ultimate strength safety factor of only 1.02 was not noticed.

At this hinge, when the tendons were slightly overstressed, their lengthening along the entire unbounded length would result in a rotation at the hinge which would shift the center of compression on the concrete blocks causing an eccentric load. A 2" change in the center of compression would more than double the stress on the blocks, resulting in failure. Hence the safety factor was very small. This was a case when the behavior of the structure was beyond the knowledge and experience of those in charge. A correct computer program adhering to the allowable stresses did not help.

4. ACCENT ON CONTROL

To further progress and to avoid problems, engineers should exercise control, not alone, but together. We should combine our capabilities in design and analyses with construction methods and schemes. When we design we should think of construction and when we construct we should think of design and analysis. Two examples will illustrate the necessity of this approach.

4.1 The Emeryville 30-story Condominium Building, Fig. 11A used a concrete frame in a heavy earthquake region. The spectrum analysis performed indicated earthquake forces 30% greater than those specified by the Uniform Building Code. The steel reinforcement is designed to remain elastic under this probable earthquake level with the design being further checked for a maximum credible earthquake greater than the 1906 San Francisco earthquake which would only produce some yielding of reinforcement at certain locations.

Heavy steel reinforcement in the beam and column joints was designed to confine the concrete, increasing its ductility. To avoid concrete congestion, the beam and column lines were offset so the rebars do not all crowd into one joint, Fig. 11B. Models of the intercepting bars and stirrups were laid out and discussed with the contractors. Thus a combined control was effectively carried out in the early stages to avoid problems.

4.2 The 539-meter 5-span Steel Arch Kuan Du Bridge, Fig. 12, is another example of the cooperation between designers, analysts, and constructors, to achieve simplicity in fabrication and economy in erection.

The design of the bridge started with an outline representing the moment diagram of a 5-span continuous beam. The arch ribs were to carry only axial force while the continuous girders along the deck would carry the live loading moments, and serve as arch ties. Since the arches are highly indeterminate and redundant, they could only be analyzed with modern computer programs.

To permit temperature expansion, the arches were free to slide on top of all piers except one, which is designed to withstand all the horizontal earthquake forces up to a certain point, beyond which the bridge supports would be contained by stoppers embedded in the piers. Hence redundancy is provided not only in the super structure but also in the piers.

Cooperation between the Contractor and the Engineer enabled the erection of all five spans in three pieces with lengths up to 685 ft. (209 m) long.

5. CONCLUSION

The above examples indicate the importance of control to insure progress and to minimize problems. Furthermore, control in the totality of structures must be exercised with engineers and constructors as a group. The group must respond to the challenges of today and tomorrow to design and build structures which are not only technologically correct and economically viable, but are also environmentally, socially and even politically acceptable.

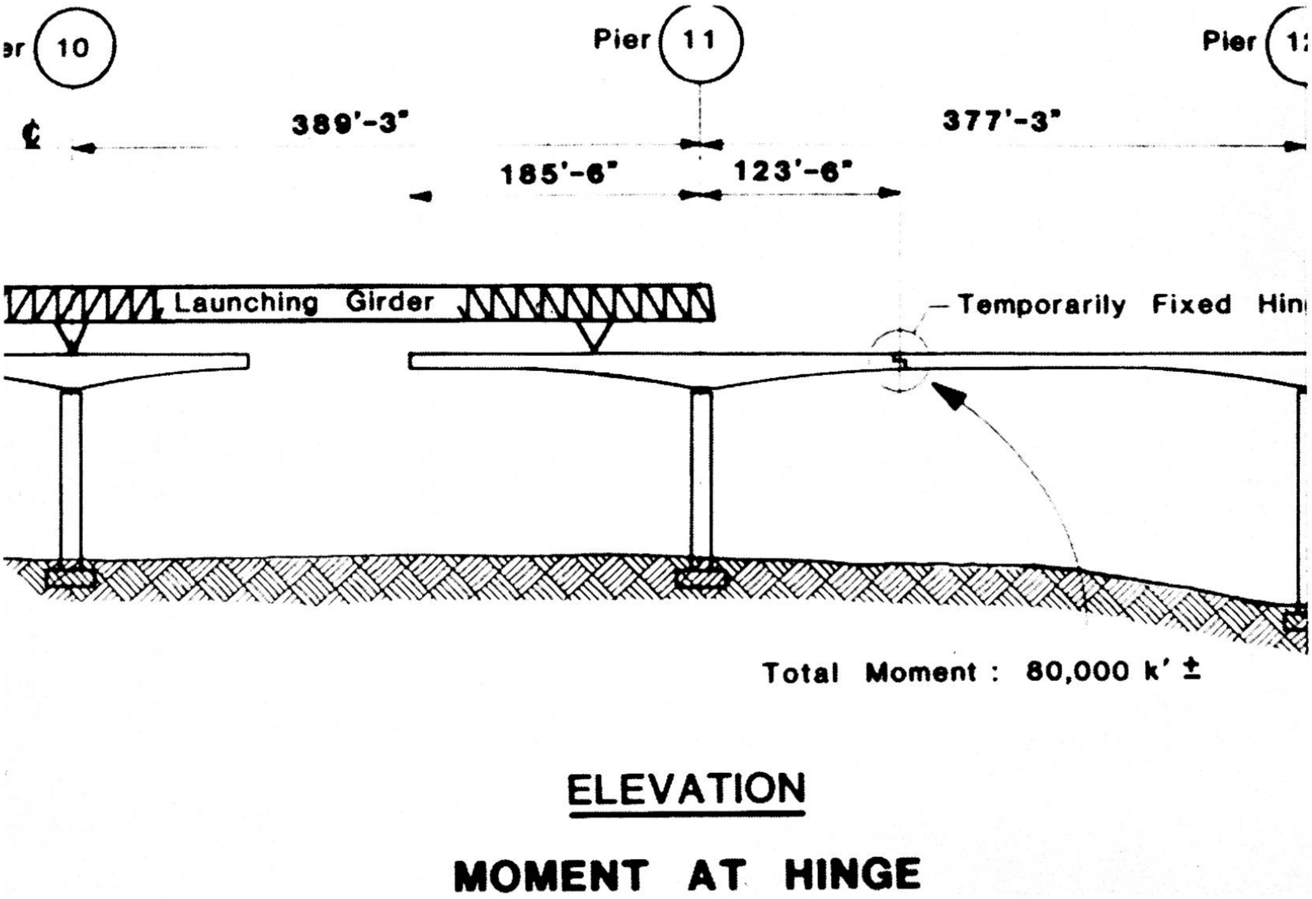


Fig. 10A

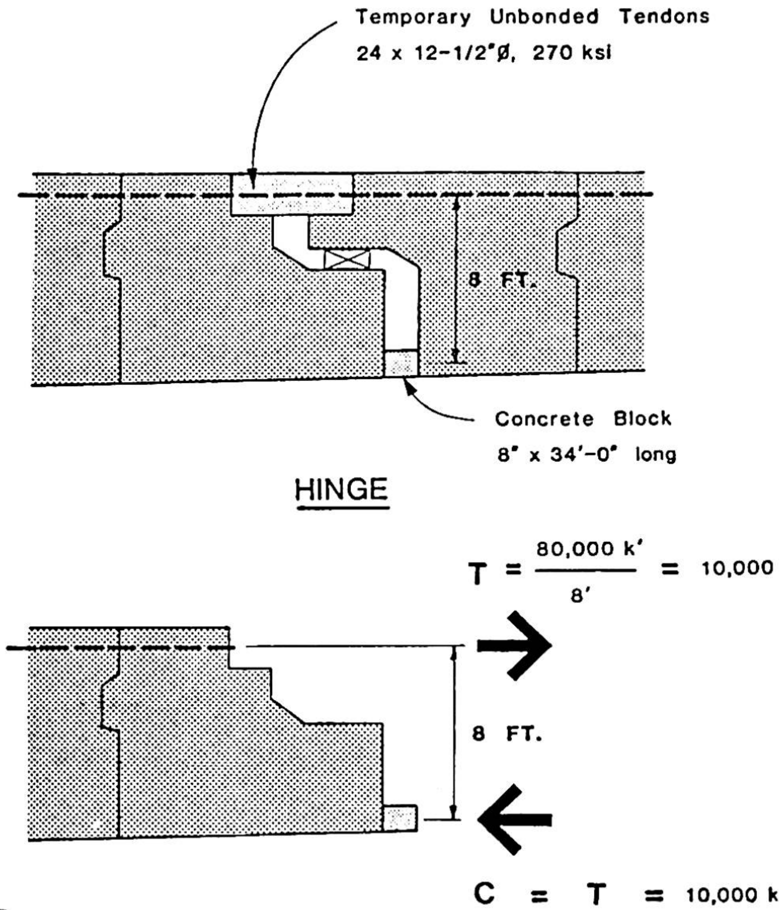


Fig. 10B

HALF FREEBODY OF HINGE

Fig. 11A

Fig. 11B

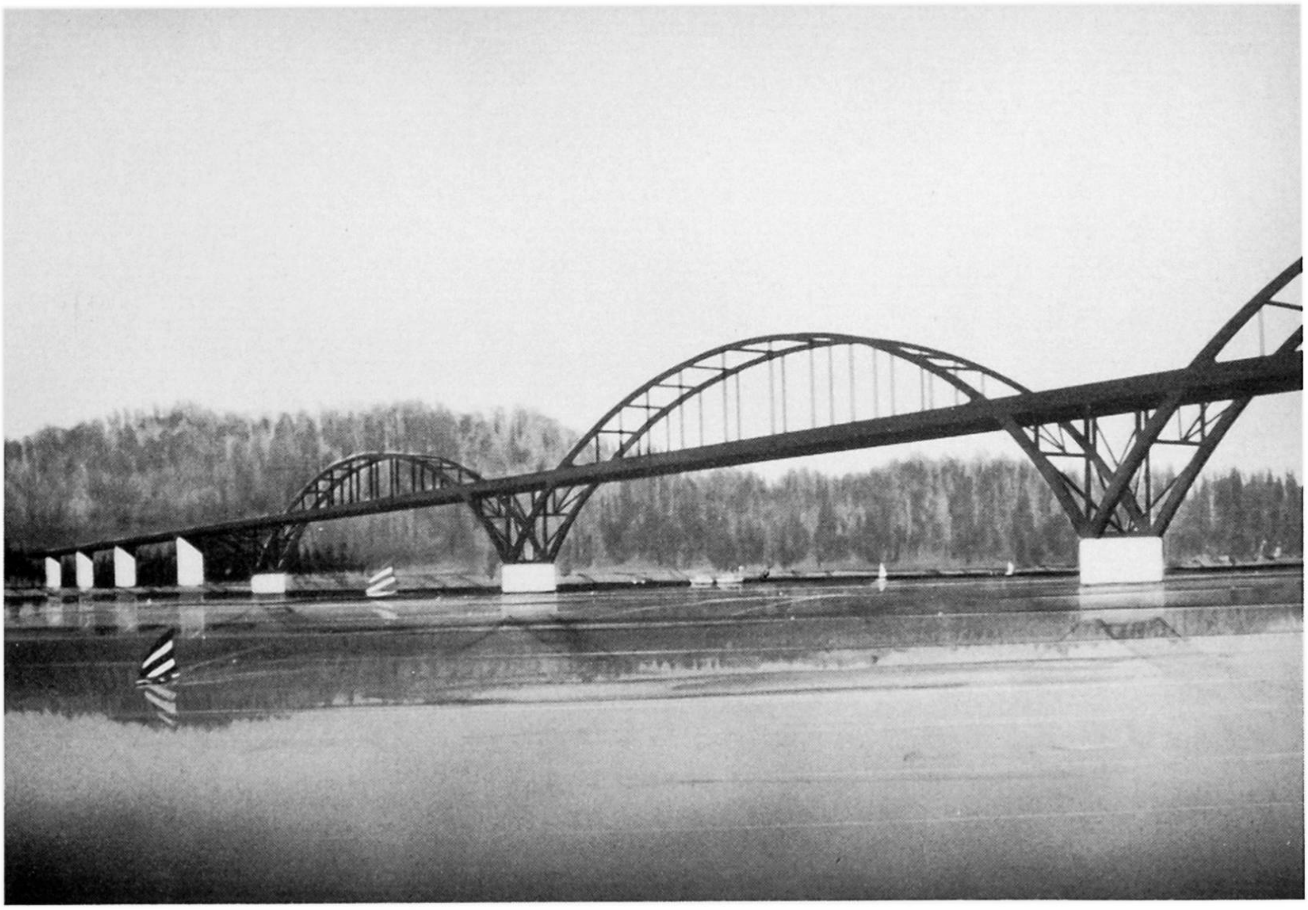


Fig. 12