

March-April 1976

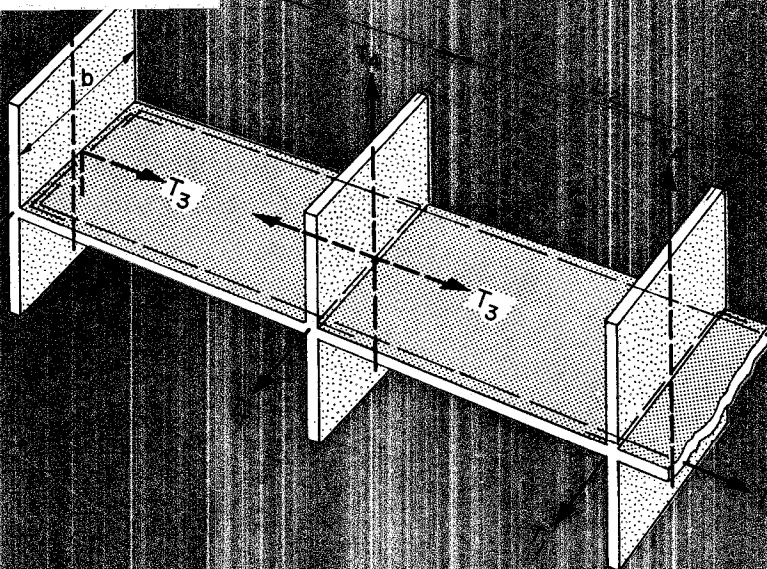
Vol. 12

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Journal

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PCI COMMITTEE REPORT ON REINFORCED BUILDINGS

Reinforced Concrete Buildings
and the Role of Prestressing
Concrete in Reinforced
Concrete Buildings

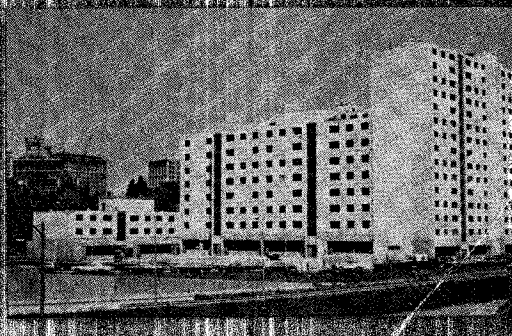
Strand Savings Using
Strain Compatibility
in Straight Design

Splicing Technique Solves
Major Precast Pile
Salvaging Operation

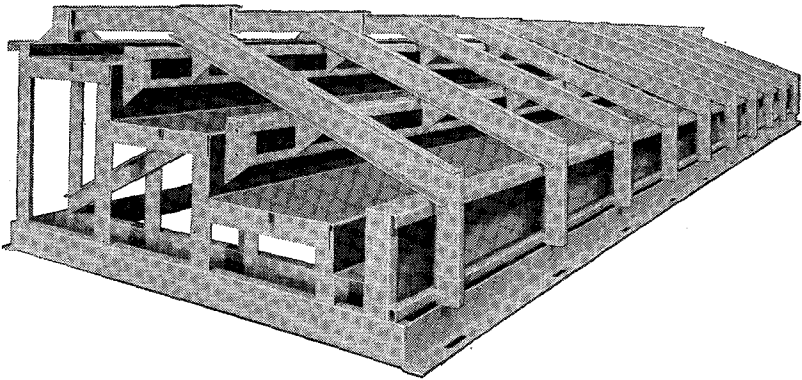
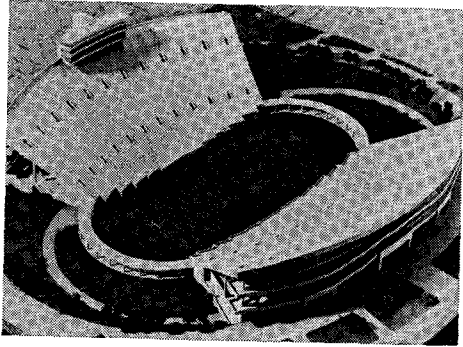
Deflection of Prestressed
Concrete Flat Plates

Reader Comments on
Recommendations for
Estimating Prestress

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Editor-in-Chief:
George D. Nasser

On the Cover: Composite of precast large-panel buildings and interaction of tie forces. For further details read the PCI Committee report on Bearing Wall Buildings starting on p. 18.

Coming in the Next Issue

Featured in the May-June 1976 PCI JOURNAL will be reports and papers on:

- ▶ Design Proposals for Reinforced Concrete Corbels
- ▶ The Prestressing of the CN Tower
- ▶ Precast Concrete Fins Stabilized With Pressure Grouted Epoxy
- ▶ A Philosophy for Structural Integrity of Large Panel Buildings
- ▶ Cyclic Tests of Welded Headed Stud Connections

Editorial Data

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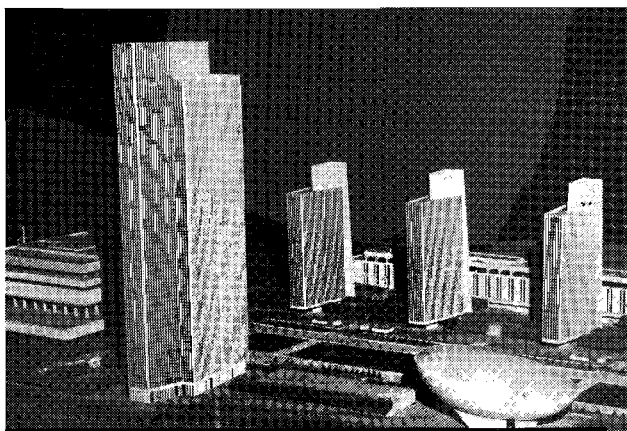
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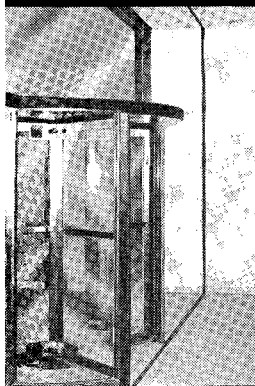
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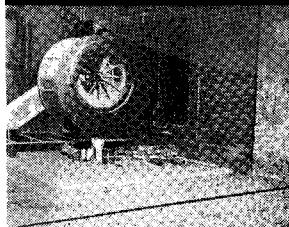
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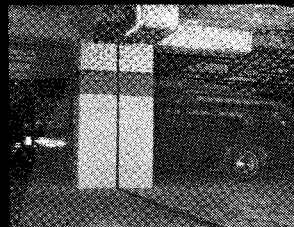


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Robert N. Bruce, Jr. and David C. Hebert

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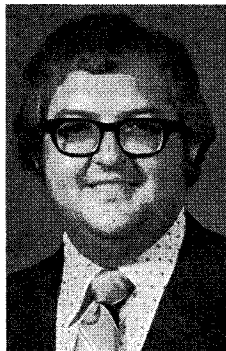
Owner: Kona Corporation. Architect: John Graham & Company. Structural Engineer: Alfred A. Yee & Associates. General Contractor: Hawaiian Dredging & Construction Company, Division of Dillingham Corporation. Precast and prestressed concrete: Dillingham Precast.

Research, application, and experience with precast prestressed bridge deck panels

James M. Barker

Structural Engineer
Illinois Department of Transportation
Springfield, Illinois

Based on his many years of experience with bridge decks and the findings of recent research, the author presents a state-of-the-art for the design, fabrication, and erection of precast prestressed concrete deck panels.



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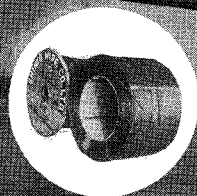
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Analysis of Lateral Load Resisting Elements—John V. Christiansen.
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Design of Load-Bearing Wall Panels—Charles H. Rath.
Treats the design considerations and designs procedures used as well as the detailed numerical calculations of a typical wall panel.

Design of Secondary Floor Members—Michael H. Barrett, Neil F. Dunbar, David D. Gillaspie.
Covers the design considerations and solutions for the design of the floor system.

Design of Frame—Richard M. Gensert, Miklos Peller, Kirit Parikh, Richard Y. Fujita.
Gives the step-by-step design calculations for the girders and columns including a commentary on the computations.

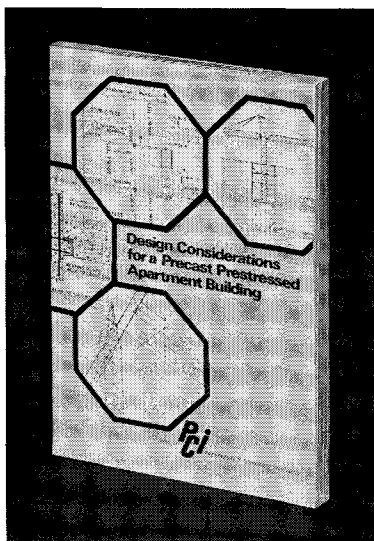
Design of Prestressed Concrete Pile Foundations—George C. Fotinos.
Discusses the various design considerations that enter into pile selection and presents the detailed design calculations for a typical prestressed pile foundation.

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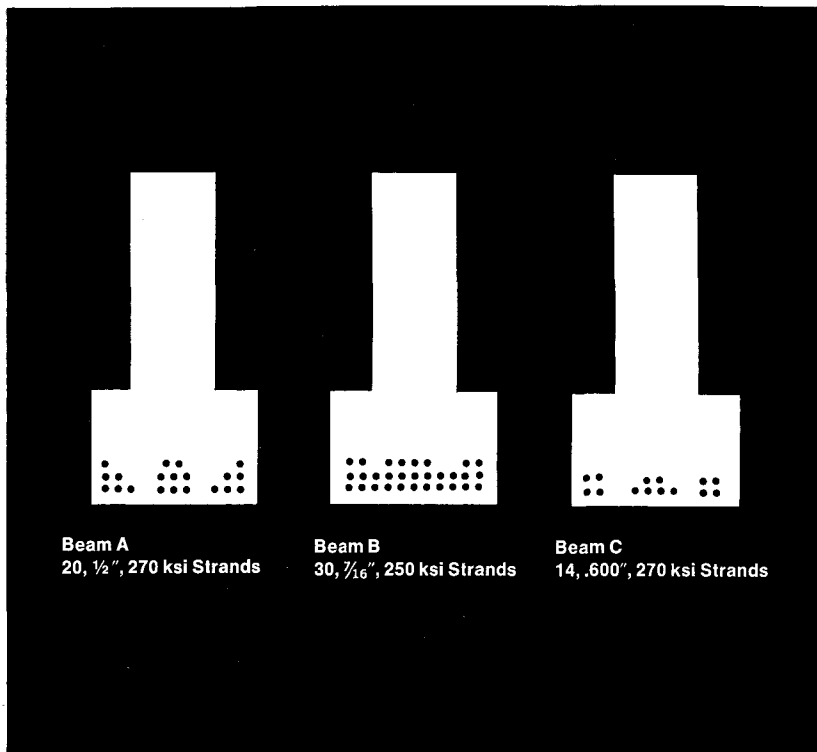
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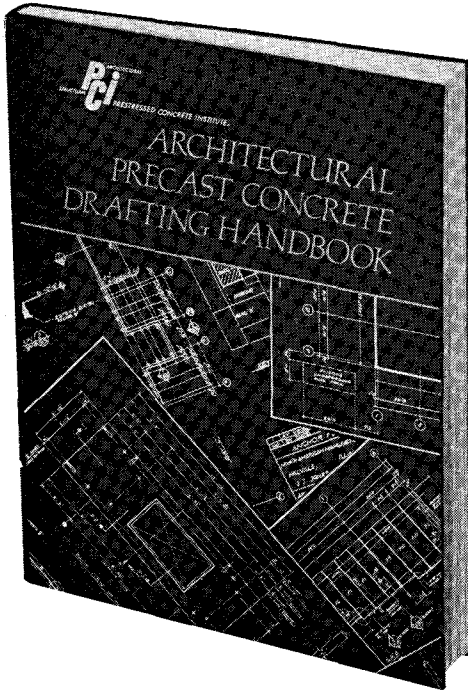
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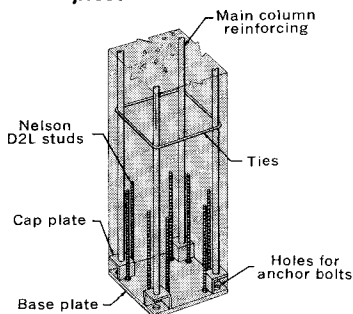
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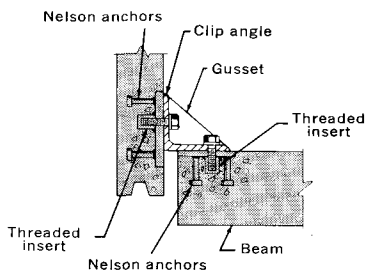
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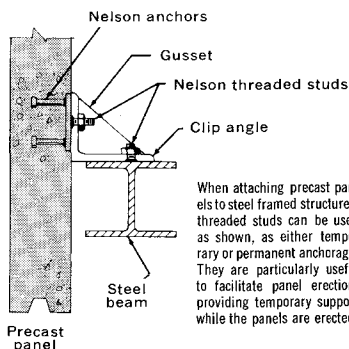
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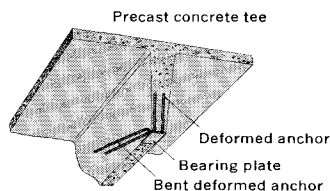
The use of Nelson D2L studs as dowels in column base plate applications effectively ties the main reinforcing bars to the base plate without the necessity for drilling holes through the base and welding the reinforcement extensively on both sides of the plate.



Studs are used for anchoring precast panels to concrete structures having either precast or poured-in-place concrete members. These methods are very compatible with in-plant casting practice. Note that the threaded inserts—also called weld nuts—are similar to those used in standard wire type screw inserts. The threaded inserts are hand welded to the insert plate over suitably sized holes in the plate.



When attaching precast panels to steel framed structures, threaded studs can be used as shown, as either temporary or permanent anchorage. They are particularly useful to facilitate panel erection, providing temporary support while the panels are erected.



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Torsion Design of Prestressed Concrete

Paul Zia and W. Denis McGee

Based on a thorough review of available research data on torsion in prestressed concrete, this report proposes an extension of the ACI Building Code provisions to cover torsion in prestressed concrete. The basic approach consists of determining the concrete contribution to the ultimate strength of a member and then proportioning reinforcement for the remaining portion of the required ultimate strength.

Formulas for the nominal torsional stress and the basic equation for torsional strength of reinforced concrete members are presented, and followed by a discussion of the torsion-shear interaction relation, taking into account the effect of the prestressing force. The requirements for both web and longitudinal reinforcements for torsion are explained.

The proposed method is illustrated by a design example of a precast prestressed L-girder in a roof framing subjected to combined torsion, bending, and shear.

(Originally published in March-April 1974 PCI JOURNAL.)

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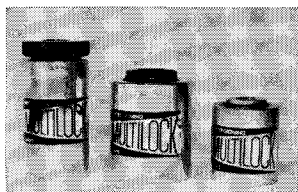
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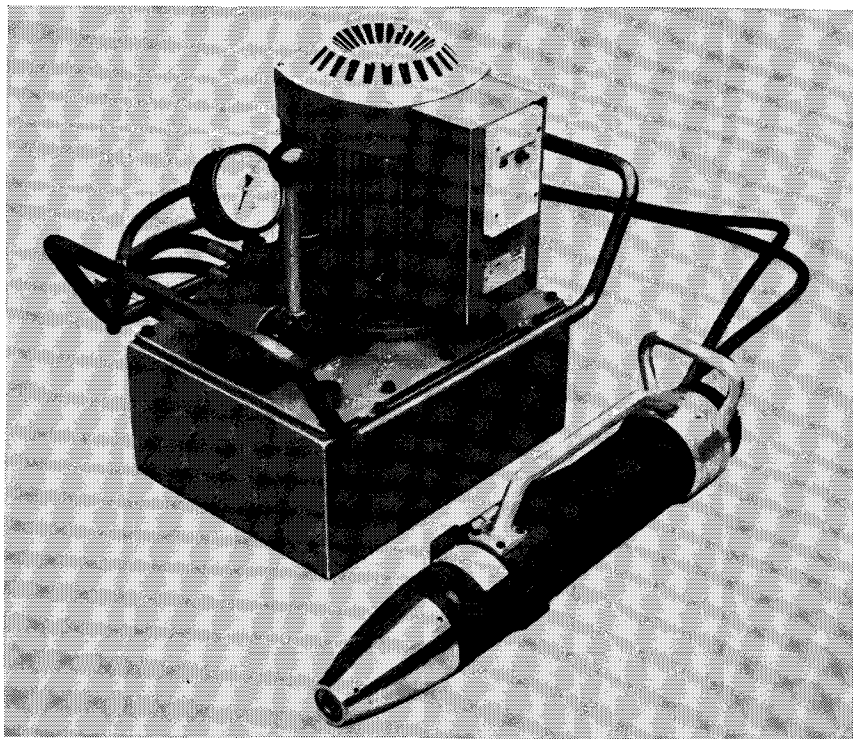
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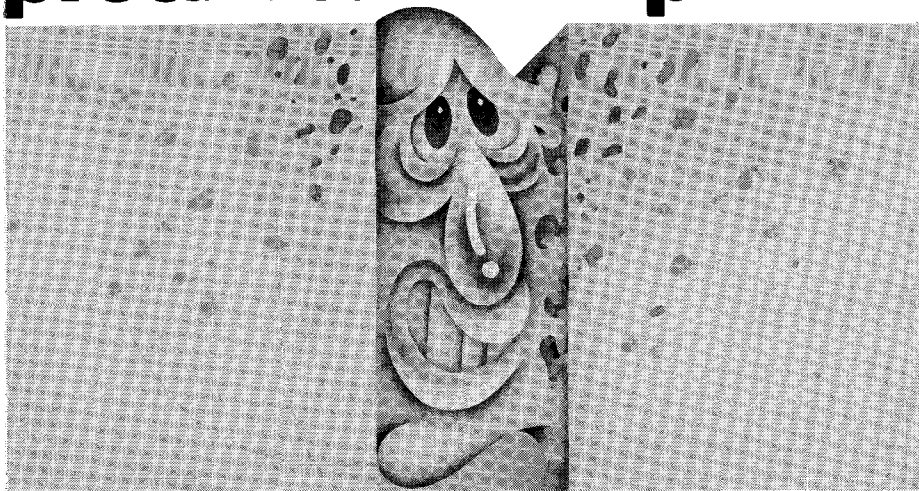
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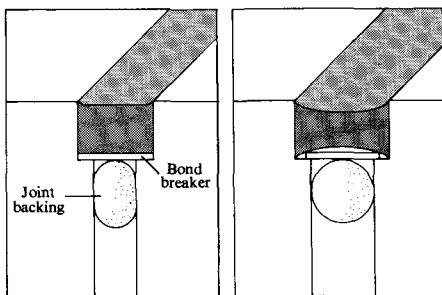
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How to avoid sealant problems when you design precast concrete panels.



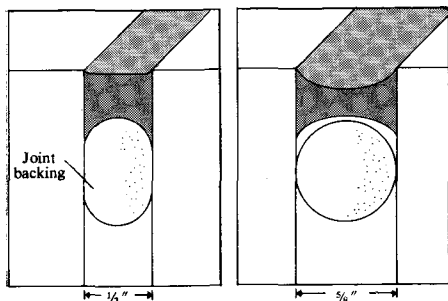
Obviously, you don't want sealant-adhesive failure in the joints between panels. And certainly we — Tremco — don't want it. So here's a brief guide to potential problem areas and tips on how to avoid them.

Let's start with design. When you're designing a joint, be sure it's wide enough to allow the sealant to move within its capabilities. If the joint is found to be too small on-site, it will have to be saw-cut to sufficient width — a costly procedure. A good rule of thumb is to design $\frac{1}{2}$ -inch wide joints for panels up to 15 feet; $\frac{3}{8}$ -



inch or wider, for larger panels. An even better rule of thumb is to consult your Tremco man while you're in the design stage.

Two ways we help. Tremco has been solving sealant problems for more than 45 years, so our man can bring a lot of experience to bear on your problems. Second, in response to the special needs of the precast industry, we've developed DYmeric®, a two-part polymer sealant designed to take the stress and movement common to precast cladding. With this kind of help, the odds are you can



avoid a lot of the following problems.

Form release agents: friend and foe. Form release agents are a necessity, but they can also create major problems for sealants.

The same action that prevents adhesion between the panel and the form can impair adhesion of the sealant bead to the joint interface. This could happen weeks or months after caulking, depending on the type of sealant, the type of release agent and the amount of joint movement.

Some release agents are less troublesome than others. However, you can only be sure of good sealant adhesion if you do two things. First, clean the joint interface just as you do the panel face, when it's removed from the form. Second, be sure the joint interface is thoroughly cleaned just before caulking. Your Tremco man can help you find the most economical way to get this done.

Don't take a powder. Another common problem that affects sealant adhesion is laitance on the joint interface. A frequent cause is the use of retarder on exposed aggregate panels. A slight change in joint design can often help prevent the retarder from migrating to the joint face.

However, it's a good idea to use the high-pressure water spray on the joint surface as well as the face of the panel, during the process of exposing the aggregate. Even then, though laitance has been successfully removed, the joint has to be thoroughly cleaned just before caulking. Since each case is different, your best bet is still to talk to your Tremco man and use Tremco DYmeric.

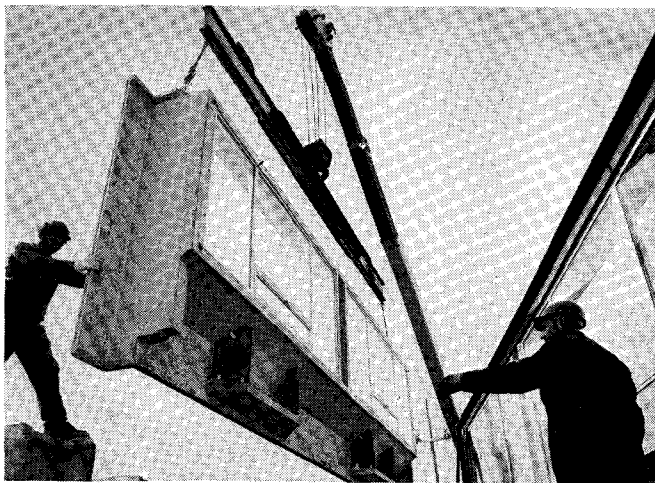
Waterproofing woes. While Tremco makes clear waterproofing coatings for masonry panels, we want to warn you

that ours, like all the others, can cause sealant failure when they're improperly used.

Our advice is to caulk first, then waterproof. But sometimes specifications call for waterproofing at the factory, to protect the panels during transit and storage. Your best bet, as always, is thorough cleaning of the joint interfaces just prior to caulking. Your Tremco man can help you decide on the right cleaning method for specific circumstances.

To sum it all up you can count on Tremco to help seal and weatherproof precast buildings better because it's the kind of thing we've been doing for more than 45 years. And you can count on us for DYmeric, the sealant designed for precast.

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President's Message



A Positive PCI

Actions internal to PCI and events subsequent to March 1, have led to (1) the honor and challenge of my appointment as interim PCI President to complete Edward Schechter's 1976 presidential term and (2) the fact that a post-tensioning division will no longer exist under the Prestressed Concrete Institute umbrella.

Concurrent with these happenings, the PCI Restructure Committee has developed concepts that will further a positive direction for PCI. Foremost is the extended emphasis of PCI as a unique organization, blending professional member and company member within one forum.

The professional members should recognize that PCI is THE Organization wherein prestressed concrete technical creativity is within an environment of interchange between them and all facets of the industry including, as before, architectural precast concrete, structural precast and pretensioned concrete, and post-tensioned concrete.

These total interests will continue to be served by the valuable PCI JOURNAL and within a strong technical and research committee structure.

In my new responsibility, I intend to insure that the restructured PCI will in fact serve the needs of ALL its members.

Eugene P. Holland

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

by
Irwin J. Speyer, P. E.
Consulting Engineer

for
**PCI Committee on
Precast Concrete Bearing Wall Buildings**

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*Members, Subcommittee on Design.

Presents design guidelines for precast bearing wall buildings subject to abnormal loadings. Emphasizes horizontal, vertical, and peripheral ties sufficient to provide interaction between all building elements. Empirical design approaches, based on various reports, are suggested and some examples are provided in the Appendix.

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CHAPTER 1—OBJECT AND RANGE OF APPLICATION

1.1 Scope of report

This report provides general guidelines for the design of precast concrete bearing wall buildings and is applicable to structures other than one- or two-family buildings.

Except as otherwise indicated in this report, these structures should conform to the latest revision of the ACI Building Code Requirements for Reinforced Concrete, ACI 318-71,¹ or the ACI Building Code Requirements for Structural Plain Concrete, ACI 322-72.³⁰

1.2 Types of construction

The following types of construction are governed by this report:

1. Precast wall panels combined with precast floors or roofs
2. Precast wall panels combined with cast-in-place floors or roofs.
3. Precast boxes or modified boxes
4. Combinations of the above.

1.3 Weight of concrete

Walls and floors may be constructed of normal weight concrete or structural lightweight concrete.

1.4 Types of panels

Wall panels may include the following types:

1. Solid concrete
2. Sandwich (insulated)
3. Ribbed
4. Hollow-core
5. Composite

1.5 Types of reinforcement

Structural elements may be reinforced with prestressing steel, welded wire fabric, reinforcing steel, or expanded metal reinforcement.

1.6 Special systems

Nothing in this report should be construed as precluding the use of any special system of large panel construction. The requirements of Section 1.4, ACI 318-71, will govern the approval of such special systems.

CHAPTER 2—DESIGN LOADS

2.1 Load requirements

Service loads, wind, and seismic requirements should be in accordance with the local applicable building code. Design loads are service loads multiplied by an appropriate load factor, determined in accordance with ACI 318-71. Since this report covers the treatment of abnormal loads, a ϕ factor of one should be assumed.

2.2 Lateral load

The structure should be designed for all lateral load requirements of the local applicable building code. As a minimum, the requirements of Section 2.2.1 should be satisfied.

2.2.1—A minimum total design force equal to 2 percent of the service dead load (weight of superstructure plus superimposed dead loads) should be applied as a lateral load at

each floor over the height of the structure. The vertical design load is taken as the service dead load. Strength and stability of the structure should be investigated for this combination of loading.

2.3 Volume changes

Effects of forces due to restrained creep, shrinkage, temperature change, and differential settlement are to be considered.

2.4 Load distribution

The stiffness and the location of the lateral load resisting elements relative to the center of rigidity of the entire structure should be considered when distributing loads.

2.5 Design loads

Forces given in this report are design loads, and are not to be factored.

CHAPTER 3—STABILITY

3.1 Bracing elements

Large panel structures should be arranged with bracing elements that provide lateral load resistance in two perpendicular vertical planes.¹⁰ Bracing elements may be loadbearing walls, non-loadbearing shear walls, or moment-resisting frames. Some possible arrangements are indicated in Chapter 5.

3.2 Ties

Vertical and horizontal tensile ties should be provided in walls, floors, and roofs of all structures over two stories in height (see Fig. 1). Ties are to be interconnected. Unless analysis indicates a larger value, the size and the spacing of these ties should be

Committee Statement

This report provides design guidance and minimum provisions which represent the best judgment of the Committee at the time the report was prepared.

The objective is to have precast concrete bearing wall buildings designed and built with a degree of continuity and ductility that will develop a reasonable resistance to the effects of abnormal loading.

It is not intended to represent that this report is the only procedure to reach the objectives; design engineers should be permitted design latitude provided they can show adequate performance.

Designers and manufacturers are invited to discuss and comment on the design considerations given in the report. Subsequently, it is intended to have the report up-dated and extended upon receipt of research data currently being developed, to consider the experience of designers familiar with this type of structure, and to formalize the recommendations in this document for building code adoption.

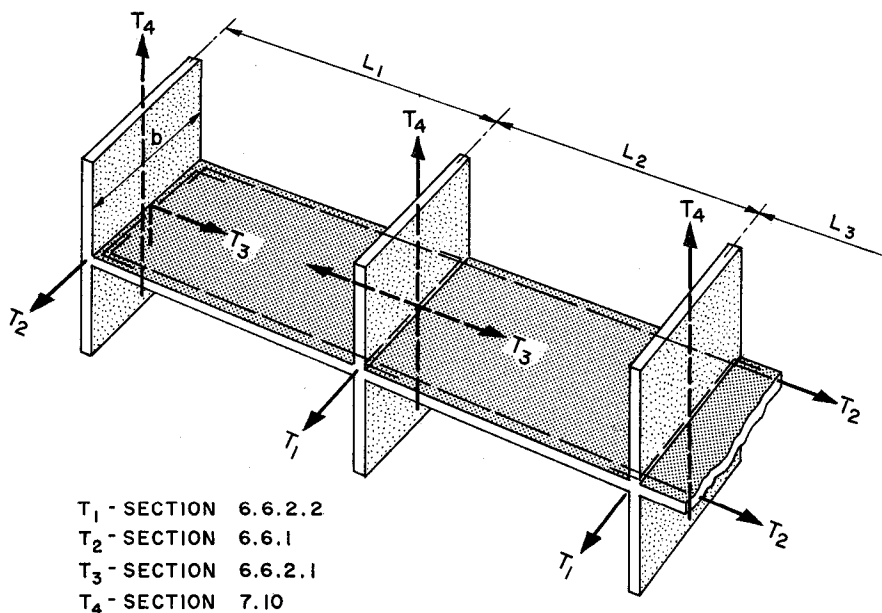


Fig. 1. Recommended tie forces in precast concrete bearing wall buildings.

determined in accordance with Chapters 6 and 7.

3.3 Continuity

Bracing elements should be continuous the full height of the structure to insure stability at all levels. Where discontinuities occur, reinforcement of local elements should insure transfer of strength across the discontinuity.

3.4 Load distribution

Where structural stability is pro-

vided by a combination of bracing walls and moment-resisting frames, consideration must be given to the possible redistribution of loads to the bracing walls resulting from the formation of plastic hinges in the frames at design loads.¹¹

3.5 Erection

Stability during erection must be considered. The contract documents should require that the Contractor review his erection procedure with a Structural Engineer.

CHAPTER 4—FOUNDATIONS

4.1 Supporting elements

Loadbearing panels and shear walls may be supported by continuous footings, isolated piers, grade beams,

or transfer girders. In determining stress distribution in both panels and supporting elements, consideration should be given to the relative flexi-

bility of the loadbearing panels or shear walls, and of their supporting elements.

4.2 Long-term effects

The sequence of construction, differential settlement, and long-term effects of creep should be consid-

ered when establishing critical loads on the substructure.

4.3 Differential settlement

Foundations are sized and detailed to minimize differential settlement of the completed structure. Differential settlement and rotation should be considered in the design.

CHAPTER 5—STRUCTURAL SYSTEMS

5.1 Types of wall panel systems

Large panel buildings are differentiated by the general arrangement of loadbearing walls (see Fig. 2):

1. *Cross wall system*—Loadbearing walls are parallel to each other and perpendicular to the longitudinal axis of the structure.

2. *Spline wall system*—Loadbearing walls are parallel to each other and to the longitudinal axis of the structure.

3. *Mixed systems*—Combination of cross wall and spline wall systems.

5.2 Box systems

Box systems, made up of walls and floor slabs, may employ tubular elements cast as integral units or individual components which are then assembled with connections to insure integral behavior. Floors may be included within the tubular ele-

ment, or they may be precast separately. Box systems are differentiated by their arrangement in the structure; they may be contiguous, or they may be separated by infill floor panels. Box systems may be arranged to form moment-resisting frames in one direction, and to act as shear walls in the perpendicular direction.

5.3 Diaphragm action

The floors of large panel structures usually serve as horizontal diaphragms, distributing applied lateral loads to the vertical bracing elements. Floors and walls may also be considered to act as moment-resisting frames.³ In this case, particular care must be taken to insure the integral behavior of the joints, including consideration for long-term creep and shrinkage. Special consideration

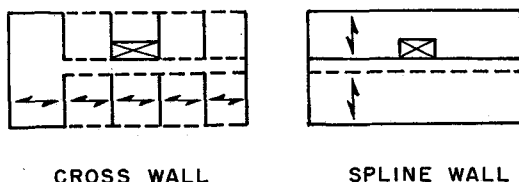


Fig. 2. Arrangement of loadbearing walls.

**POSSIBLE DIFFERENTIAL
CAMBER OF FLOOR SLABS**

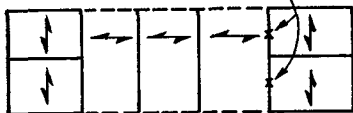


Fig. 3. Floor spans in different directions.

should be given to the interconnection of precast floor elements in Seismic Zones 2 and 3.³⁴

5.4 Structural layout

In choosing the structural layout of panels, consideration should be given to arranging the direction of floor spans so that gravity loads of the floors resist uplift caused by lateral loading. Such an arrangement will require special attention to the possible differential camber of adjacent floor panels spanning perpendicular to each other (see Fig. 3).

5.5 Expansion joints

Expansion joints may be located as required by local climatic conditions, by the arrangement of stiffen-

ing elements, and as necessary to accommodate effects of shrinkage and creep. In general, they should not be spaced more than 180 ft unless special consideration is given to the effects of creep, shrinkage, and thermal conditions. Expansion joints should also be considered at abrupt changes in the structure.

Generally, expansion joints separate the superstructure for its full height. Additional localized expansion joints may be required, as determined by the characteristics of the structure. Expansion joints in Seismic Zones 2 and 3 should be arranged to prevent hammering.

5.5.1—Special attention should be given to portions of buildings which may be subject to extreme differential temperature changes (rooms containing high temperature equipment, exterior portions of structure attached to interior portions, etc.).

5.5.2—Loadbearing elements at expansion joints should be connected to the fixed side floor or roof slab in accordance with the requirements for horizontal ties in floors and roofs, as given in Chapter 6.

CHAPTER 6—FLOORS AND ROOFS

6.1 Cast-in-place members

Cast-in-place floors and roofs should be designed and constructed in accordance with ACI 318-71.

6.2 Precast elements

Precast floor and roof elements may consist of solid slabs, hollow-core slabs or ribbed units, prestressed or conventionally reinforced. These elements are designed in accordance

with ACI 318-71 and the following additional requirements:

6.2.1—If intended to provide diaphragm action, precast elements with continuous grouted keyways between the elements may be considered fully effective when floor ties are provided in accordance with this report. Mechanical connections may be used in lieu of, or in combination with, grouted keyways. A structural

cast-in-place concrete topping, properly reinforced and bonded to the precast elements, may also be considered to act as a diaphragm in combination with the precast elements.

6.2.2—Precast floor and roof elements with minimal or no top reinforcing steel at the supports should have a sufficient bearing length to safely transfer applied loads by direct bearing, but in no case less than:

1. 2½ in. for solid and hollow-core slabs.

2. 3½ in. for ribbed slabs, including solid slabs with projecting nibs.

The length of bearing is measured from the face of the wall, except that, if the wall is chamfered more than ½ in., the bearing length is measured from the inside edge of the chamfer. The above minimum bearing lengths may be reduced if bearing surfaces are adequately armored. For this special condition, the usual manufacturing and erection tolerances may require modification, as determined by the Engineer.

6.2.3—Precast floor and roof elements, with sufficient top reinforcement at the supports to resist calculated support moments, may have reduced bearing lengths, provided the joint is designed to transfer floor loads to the supporting bearing panels. Permissible bearing length may be determined by analysis or tests. Consideration should be given to the possible need for bottom reinforcement across the joint to prevent separation due to creep and shrinkage, and the subsequent loss of contact of precast element and joint fill concrete.

6.3 Tolerances

Manufacturing and erection toler-

ances should be considered in determining the bearing lengths used in design. The contract documents should indicate the tolerance required by the Engineer of Record.

6.4 Restraint

For simple spans, the effect of a possible clamping action of wall panels on precast floor elements, and the resulting restraint against translation and rotation, should be considered. Continuity of precast floors and roofs may be considered if the following requirements are satisfied:

1. Negative reinforcement, as determined by elastic analysis, is spaced not greater than three times the floor or roof thickness.

2. Floor tie requirements of Section 6.6 are provided in addition to the negative reinforcement as determined by analysis.

6.5 Ties

Horizontal tensile ties should be provided in floors and roofs, or within the joints of precast elements, to satisfy all loading requirements, and to perform the following functions:

1. Resist forces which may be induced by erection eccentricities.

2. Transfer to floor diaphragms the horizontal reactions due to lateral loads on wall panels.

3. Transfer horizontal reactions due to applied lateral loads on shear walls.

4. Resist tensile forces due to restrained volume changes, differential settlement, and lateral loads.

6.6 Minimal tie requirements

The following minimal requirements for horizontal floor and roof ties (Fig. 1) must be satisfied. Tie capacity is based on yield stress.

6.6.1—At each floor and roof level a continuous peripheral tie, T_2 , capable of resisting a design force sufficient to develop diaphragm action, but not less than 16,000 lb, should be provided within the depth of floor or roof element, or sufficiently close to these elements and connected thereto. The tie may be reinforcing steel in the grout joint or continuous reinforcing steel (including prestress) in an edge beam. Reinforced spandrels or walls adequately anchored to the floor or roof may also be considered.

If ties are interrupted by offsets in the structure, supplemental reinforcement properly spliced can be provided to insure effective continuous tie action. Where buildings are provided with expansion joints, each section must be treated as an entity. A precast floor or roof slab may be considered to be an edge beam, provided the continuous tie reinforcement is located within 4 ft of the perimeter of the structure.

6.6.2—At each floor and roof level, continuous longitudinal and transverse internal ties should be provided at approximately right angles across the building. Ties may be mechanically anchored or spliced to the peripheral reinforcement at each end. At openings in floors or roofs the total tie force must be maintained by supplemental reinforcement, properly anchored. Longitudinal and transverse ties should be in accordance with the following:

6.6.2.1—Longitudinal ties, T_3 (in the direction of the floor span), connecting floor or roof elements that abut over internal walls, or connecting external bearing walls with floor or roof, should be capable of resisting a design force of 2½ percent of

the service load on the wall, but not less than 1500 lb per lin. ft of distance measured perpendicular to the span. These ties should be spaced no greater than 8 ft on centers. They may project from the precast floor or roof slabs, or be embedded within joints between the precast elements.

Reinforcement must be of sufficient length and with sufficient concrete cover on all sides to develop the specified design force. When so developed, the tie force beyond the end of this reinforcement may be considered to be provided by the normal reinforcing steel in the floor or roof slabs. Longitudinal ties at exterior bearing walls, or other bearing walls which may be loaded from one side only, can develop the specified forces by projecting wall reinforcement into the floor, or by providing a properly designed mechanical anchorage between floor and wall.

End walls may also be braced by structural walls or frames perpendicular to these end bearing walls. The bracing elements may comprise a stair tower, elevator tower, or similar components. When such bracing elements are used, connections between the bracing elements and the bearing walls should be sufficient to resist a design force of 2½ percent of the service load on the bearing wall but not less than 1500 lb per lin. ft of distance measured perpendicular to the span at each floor level.

6.6.2.2—Ties, T_1 , transverse to the direction of the floor or roof span should be capable of resisting a design force not less than 1500 lb per lin. ft of distance measured parallel to the direction of floor or roof span. These ties may be encased in the

floor or roof units, may be placed in a structural topping of thickness not less than 2 in., or may be concentrated at or in the transverse walls. Reinforcement located in the transverse walls may be placed within the depth of the floor, or at a distance from the floor not greater than the floor thickness.

6.6.2.3—Flexural and temperature reinforcement provided within floor or roof slabs may be considered to provide part or all of the above specified forces when properly detailed and anchored.

6.7 Shear keys

For precast floor or roof slabs with

properly grouted shear keys or mechanical connections,⁴ the line load from partitions parallel to the direction of the floor span may be assumed distributed in accordance with the lesser of the following:

1. Distribution as determined by structural analysis, considering flexural strength and shear strength in the lateral direction.

2. One-half the clear span length of the floor.

Note: All values recommended in this chapter may be varied in accordance with the procedures of Section 1.6.

CHAPTER 7—LOADBEARING WALLS

7.1 Design requirements

A wall is here defined as a vertical loadbearing element whose length is equal to or greater than four times its thickness.⁴

The vertical load-carrying capacity of walls may be determined in accordance with ACI 318-71, Chapter 10 or Chapter 14.

Loadbearing walls whose thickness is at least 1/25 of its unsupported height may be designed in accordance with the requirements of ACI 322-72.

7.2 Bracing

Loadbearing panels should be braced. This bracing may take the form of transverse walls directly connected to the loadbearing panel, or connection of the loadbearing panel through floor diaphragm action to transverse walls located re-

mote from the panel.²

When applying the design procedures of ACI 318-71, the value of k may be determined in accordance with the following:

7.2.1—Unless analysis indicates that a smaller value is acceptable, the value of k should be taken as one for panels braced against side-sway.

For panels braced against side-sway, the effect of slenderness may be neglected when

$$kL/r < 34 - 12 (M_1/M_2) \quad (1)$$

where

k = effective length factor for compressive members

L = height of panel between supports

M_1 = value of smaller design end moment on panel, positive if bent in single curvature,

negative if bent in double curvature

M_2 = value of larger design end moment on panel, always positive

r = radius of gyration

7.2.2—For panels braced against sidesway, and connected on all four edges, the value of k may be taken as:

$$k = 1 \text{ for } (L/b) < \frac{1}{2} \quad (2)$$

$$k = 1\frac{1}{2} - (L/b) \text{ for } \frac{1}{2} \leq (L/b) \leq 1 \quad (3)$$

$$k = \frac{1}{1 + (L/b)^2} \text{ for } (L/b) > 1 \quad (4)$$

where b is the length of panel between supports.

7.2.3—For panels braced against sidesway, and connected on one vertical edge and at top and bottom, the value of k may be taken as:

$$k = 1 \text{ for } (L/b) < 1 \quad (5)$$

$$k = 1 - 0.423[(L/b) - 1] \quad (6)$$

for $1 \leq (L/b) \leq 2$

$$k = \frac{1}{[1 + 0.5(L/b)^2]^{\frac{1}{2}}} \quad (7)$$

for $(L/b) > 2$

7.2.4—For panels braced against sidesway, and connected at top and bottom only, the value of k should be taken as one.

7.3 Connections

In order to be considered as effectively braced along an edge, the loadbearing panel must be adequately attached to another component which is approximately at right angles to the bearing wall, and which has a length not less than one-fourth of its clear height between floors. Equivalent rigidity may be provided by other acceptable methods.

Attachment between walls may be achieved by providing a mechanical connection (e.g., bolting or welding), or by constructing a reinforced concrete joint (Fig. 4). The connection should be able to resist the design force (factored dead load plus live load) determined by static analysis, plus a bracing force of 2½ percent of this vertical design force carried along the length of lateral support.

If loadbearing panels are pierced with significant openings, the portion between such openings should be considered as unbraced along the vertical edges bounded by the openings.

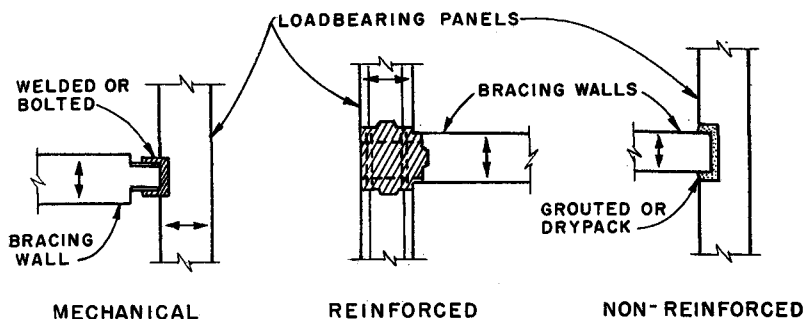
7.4 Effective thickness

The effective thickness of loadbearing walls for purpose of design may be determined in accordance with the following criteria:²⁰

7.4.1—The effective thickness of solid concrete panels without a surface treatment is taken as the full thickness.

7.4.2—The effective thickness of solid concrete panels cast monolithically with a surface facing having a compressive strength equal to or greater than the concrete backup of the panel is taken as the total panel thickness less the average depth of the reveal of any exposed aggregate, when the average depth of the reveal exceeds 3 percent of the total panel thickness.

7.4.3—The effective thickness of solid concrete panels that have a separate facing, not cast monolithically with the panel, is taken as the total panel thickness less the facing thickness, unless adequate means are provided to insure interaction.



NOTE: ARROWS INDICATE MOTION WALLS ARE BRACED AGAINST

Fig. 4. Typical wall-to-wall connections (plan views).

7.4.4—The effective thickness of ribbed panels cast monolitically may be conservatively taken as three times the radius of gyration of the panel in the direction in which stability is being considered.

7.5 Flange width

The effective flange width of symmetrical ribbed panels should not exceed the least of:

1. One-fourth the height of the panel between supports.
2. Eight times the thickness of the flange on either side of the stem.
3. The center-to-center spacing of the stems.

The effective projecting flange width for unsymmetrical panels should not exceed the least of:

1. One-twelfth the height of the panel between supports.
2. Six times the thickness of the flange on either side of the stem.
3. One-half the clear distance to the next rib.

7.6 Hollow-core panels

Hollow-core panels should be designed on the same basis as solid panels, with due consideration given

to the voids in determining section properties and strength.

7.7 Sandwich panels

Sandwich panels consisting of two layers of concrete separated by a non-structural insulation core should have the two layers sufficiently interconnected, preferably by mechanical shear ties (Fig. 5), so that both layers can be considered structurally effective. Shear stresses are not to be assumed resisted by the non-structural insulation core unless confirmed by test. Compressive and flexural stresses are assumed to be carried by the concrete only.

The spacing of ties should be such that the slenderness ratio of the concrete layer supporting load does not exceed the slenderness ratio of the

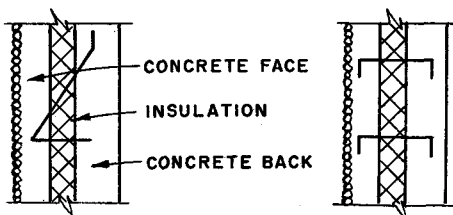


Fig. 5. Ties in sandwich panels.

overall panel, unless it can be demonstrated that a larger spacing will not affect the overall behavior of the panel.

Where the outer layer of concrete is not directly supported on the panel below, the ties must be designed to transfer the weight of this layer through the core to the loadbearing layer. Consideration must also be given to the effects of differential shortening between loaded and unloaded layers.

When vertical loads are applied to one layer only, it should be assumed that this layer resists the total applied load. When considering the effect of slenderness for this case, the overall thickness of the sandwich panel may be used provided ties and/or bond are sufficient to insure composite action.

7.8 Reinforcement

Wall panel reinforcing practice varies. Experience indicates that the following general guidelines are appropriate:

7.8.1—Casting and handling methods chosen to minimize the flexural tensile stresses are more effective than reinforcing steel in the control of cracks. The calculated tensile stress on the effective concrete section should preferably not exceed $5\sqrt{f'_{ci}}$, where f'_{ci} is the concrete compressive strength at time of stripping.

7.8.2—Areas of abrupt change in the cross section of the panel should be reinforced. For panels less than 6 in. thick, provide at least one No. 4 bar around openings. For panels 6 in. and thicker, provide at least two No. 4 bars.

Bars should be continuous around

openings, or extend 2 ft past openings on all sides which is sufficient to develop the bar. Other sizes and arrangement of reinforcement, which has been demonstrated as sufficient to prevent cracking, may be used.

7.8.3—Loadbearing walls should be reinforced with an amount of steel of at least $0.001bL$, both vertically and horizontally, unless designed in accordance with ACI 322-72. Spacing of reinforcement shall not exceed 30 in. for interior walls or 18 in. for exterior walls.

7.8.4—At points of significant local bearing stress, panels should be reinforced against splitting. Design may be in accordance with Part 6 of the *PCI Design Handbook*.³³

7.9 Concentrated loads

Concentrated loads on wall panels should conform to the following:

7.9.1—Concentrated loads may be assumed to be distributed uniformly within a zone as indicated in Fig. 6.

7.9.2—Bearing stresses under concentrated loads should not exceed the values determined by the methods of ACI 318-71, Section 10.14, or ACI 322-72, Section 7.10, whichever document is used for the design of the panels.

7.9.3—For uniform bearing on unreinforced concrete, without provi-

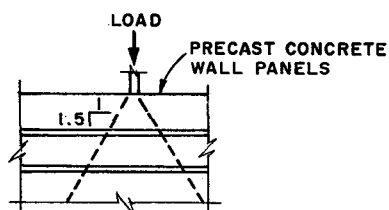


Fig. 6. Distribution of concentrated loads on walls.

sion for confinement reinforcing, the design provisions of the *PCI Design Handbook*, Section 6.1.6, will govern.

7.9.4—For uniform bearing on concrete with confinement reinforcing, the design provision of the *PCI Design Handbook*, Section 6.1.9, will govern.

7.9.5—Where loads are distributed across vertical joints in panels, design of the joint must provide for the transfer of these loads.

7.10 Buildings over two stories

In buildings over two stories in height, continuous vertical ties (T_4 in Fig. 1) should be provided from foundation to roof in all loadbearing walls and structural shear walls. The capacity of these ties, based on yield stress, must be sufficient to resist any calculated tensile force (uplift, shear-friction), but should, as a minimum, provide a design capacity of 3000 lb per lin. ft of wall. A minimum of two ties should be provided per wall panel. Where panels have significant openings, consideration must be given to the possible need for additional vertical ties between such openings.

7.11 Design eccentricities

Loadbearing walls which are braced laterally may be designed assuming no moment transfer to the floors, unless the connection between wall and floor has been designed and reinforced to transmit moments.

In establishing design eccentricities, consideration should be given to:

1. Erection tolerances
2. Manufacturing tolerances
3. Joint details

4. Lateral forces, such as wind and seismic

The minimum total eccentricity is 0.6 in. (see ACI 318-71¹).

7.11.1—The resultant eccentricity of applied loads, with respect to the neutral axis of the panel at the point under consideration, may be determined using Reference 3:

$$e_a \text{ (or } e_b) = \frac{P_1 e_1 + P_2 e_2}{P_1 + P_2} \quad (8)$$

where

P_1 = design loads from wall above, considering level "a" or level "b"

e_1 = eccentricity of P_1

P_2 = design loads from floor at level under consideration

e_2 = eccentricity of P_2

e_a = eccentricity at top of panel

e_b = eccentricity at bottom of panel

7.11.2—When employing the design procedures of ACI 322-72, the design value of eccentricity, e , shall be taken as the average of e_a and e_b (see Fig. 7).

7.11.3—The value of e_1 is a function of the manufacturing and erection tolerances, and the location of the bearing surface of the upper

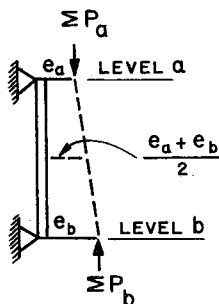


Fig. 7. Average eccentricity on a wall panel.

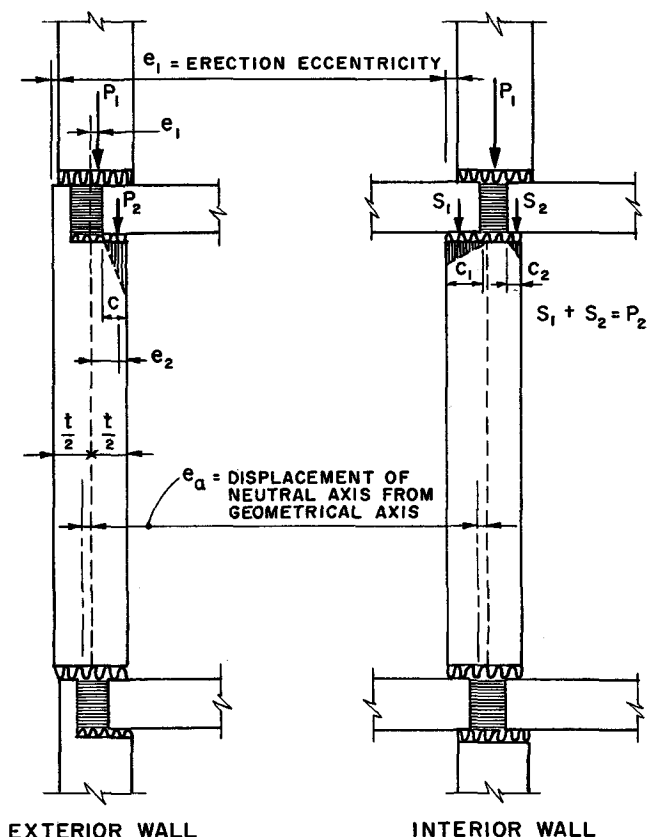


Fig. 8. Development of eccentricities in wall panels.

wall panel on the joint (see Fig. 8).

7.11.4—The value of e_2 may be determined by statics:

For wall panels loaded on both sides

$$e_2 = \frac{S_1(t/2 - c_1/3) - S_2(t/2 - c_2/3)}{S_1 + S_2} \quad (9)$$

For wall panels loaded on one side

$$e_2 = t/2 - c/3 \quad (10)$$

where

S_1, S_2 = floor loads

c_1, c_2 = bearing length of loads S_1 and S_2

t = thickness of panel

7.12 Shear transfer

Wall panels which are intended to act as beams are designed in accordance with the methods of ACI 318-71. Where more than one-story height of wall is intended to act integrally, provision must be given to the transfer of horizontal shears at the joints by means of positive interconnection of panels.

7.13 Transfer girders

The design of wall panels supported by transfer girders should consider the effective stiffness of the connected elements. One design method is given in Appendix 1.

CHAPTER 8—SHEAR WALLS

8.1 Ductility

Large panel structures constructed in accordance with this report should be considered nonductile, unless they are continuously reinforced for their full height, in accordance with Reference 16, or are post-tensioned to insure ductile behavior.

8.2 Lateral resistance

The resistance to lateral load in large panel structures is usually provided by shear walls, which may also be loadbearing walls. Lateral resistance may be obtained by the combined action of shear walls and moment-resisting frames, or by connecting walls and floors to form moment-resisting frames. In the latter case, special precautions should be taken to insure the integrity of the joint, in accordance with Section 5.3.

8.3 Bracing

Shear walls may be considered as braced at each floor when the floor and walls are tied in accordance with Section 6.6. When shear walls

extend beyond the edge of the floor (Fig. 9), the projecting portion may be considered as braced provided that:

1. The floor is tied to the wall in accordance with this report.
2. The projecting portion considered as braced does not exceed twelve times its thickness.

8.4 Transverse walls

Shear walls may be assumed to act together with transverse walls to form a monolithic flanged section when properly connected to transmit joint stresses (unit shearing stress, VQ/I) plus the effects of differential shortening.

Where interior and exterior walls intersect, consideration must be given to the effect of temperature differential.

The effective length of flange wall on each side of a shear wall (Fig. 10) is taken as the least of the following, unless a more accurate analysis, including the effect of shear lag, is made:

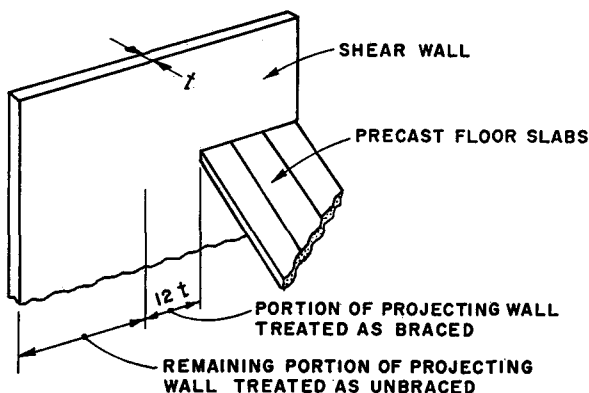


Fig. 9. Bracing beyond floor edges.

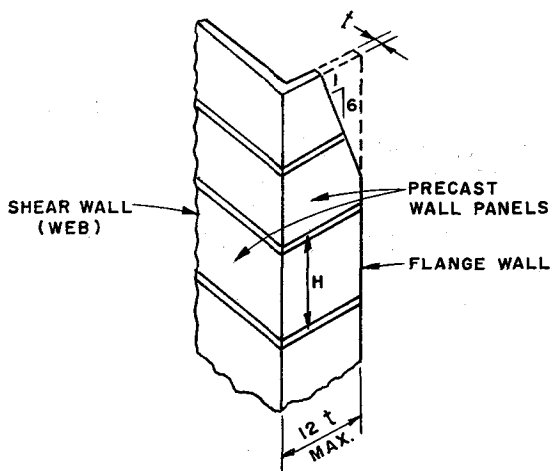


Fig. 10. Effective limits for flange walls.

1. The distance from the shear wall to any significant opening in the flange wall.

2. One-sixth the distance from the level under consideration to the top of the flange wall.

3. Twelve times the thickness of the flange wall.

8.5 Openings

Shear walls with large openings should be designed as coupled walls, considering the stiffness of the connecting beam.^{15,21}

8.5.1—Walls so coupled should be in the same plane. The analysis should also consider the support conditions at the base of the wall.

8.5.2—Beams designed to couple shear walls should be reinforced for flexure and shear. The total design shear in the beam should be taken by closed stirrups, with no allowance for the shear resistance of plain concrete. Distribution of flexural reinforcement should be in accordance with ACI 318-71, Section 10.6.

8.5.3—The effect of the floor in

contributing to the stiffness of the beam may be considered when the floor is cast-in-place or of composite construction and properly connected to the shear wall and to the beam; otherwise it should be neglected.

8.5.4—Proper coupling of in-plane walls will significantly reduce their deflection. When a cast-in-place connection between walls is provided, the effect of reduced rigidity due to any cracking of the connection should be considered.

8.6 Flexural strength

The flexural strength of shear walls¹⁷ may be determined in accordance with the following:

8.6.1—Vertical reinforcing steel concentrated at the ends of the wall. For this condition, the usual theory of flexure is applicable.

8.6.2—Vertical reinforcing steel distributed along the length of the wall. For this condition, a strain compatibility analysis is required to determine flexural strength. In lieu of detailed analysis, the following

equation may be used:

$$M_u = (\phi/2) A_s f_y L_w \left[1 + \frac{N_u}{A_s f_y} \right] \times \left[1 - \frac{c}{L_w} \right] \quad (11)$$

where

M_u = design resisting moment of the wall

A_s = total area of vertical reinforcement

f_y = specified yield strength of reinforcement

L_w = horizontal length of shear wall

N_u = design axial load, positive if compression

c = distance from extreme compression fiber to neutral axis

ϕ = capacity reduction factor, here taken as one

The value of c/L_w may be determined from the following:

$$\frac{c}{L_w} = \frac{\frac{\rho_v f_y}{f_c} + \frac{N_u}{L_w t f_c}}{\frac{2\rho_v f_y}{f_c} + 0.5\beta_1} \quad (12)$$

where

t = thickness of wall

β_1 = factor defining rectangular stress block (ACI 318-71, Section 10.2.7)

$\rho_v = A_s/(L_w t)$

f_c = specified 28-day concrete compressive strength

8.7 Shear strength

The shear strength of walls may be determined in accordance with ACI 318-71, Section 11.16.

8.8 Uplifts

Depending on their location, shear walls may also be loadbearing walls.

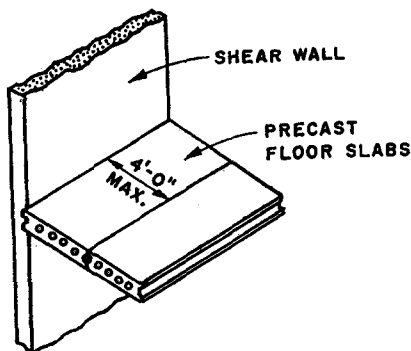


Fig. 11. Effective floor load on parallel shear wall.

When shear walls are not bearing walls, uplifts may occur under lateral loading. It is often advantageous to "load" these walls to counteract the calculated uplifts. One method is to post-tension the walls. Other methods are to tie the parallel floor span edge to the shear wall, or to tie the shear wall to a loadbearing wall.

For slabs spanning parallel to the wall and connected thereto, it may be assumed conservatively that a width of floor not in excess of 4 ft or one slab width, whichever is less (Fig. 11), contributes load to that part of the shear wall exhibiting uplift, provided the connection between wall and floor is sufficient to transfer such loads.

8.9 Stiffness

Unless a stability analysis includes the effects of lateral deflection ($P-\Delta$ effect), the stiffness of all lateral load-resisting elements within a structure (in the direction considered) must be such that the following criteria are satisfied:⁶

$$H\sqrt{P/(EI)} \leq 0.6 \text{ for } n > 4 \quad (13)$$

$$H\sqrt{P/(EI)} \leq 0.2 + 0.1n \quad (14) \\ \text{for } 1 < n \leq 4$$

where

H = total height of structure

n = number of stories in structure

P = sum of all vertical service loads on structure

EI = sum of bending stiffness of all restraining elements in direction considered, assuming the sections are uncracked

The units in Eqs. (13) and (14) must be consistent.

8.10 Lateral deflection

Lateral deflection (drift) under ser-

vice loads should not exceed $H/600$.

The differential lateral deflection between adjacent walls resisting lateral loads must be limited so as not to cause distress of any member attached to these walls. Otherwise, these members must be designed so that they can tolerate the differential movement.

Any reduction in wall stiffness due to the characteristic behavior of joints within that wall (both horizontal and vertical joints) should be considered in calculating lateral deflection (see Chapter 9).

CHAPTER 9—JOINERY

9.1 Load transfer

Properly designed and constructed joints are an important factor in assuring load transfer without distress, and in achieving stability of large panel structures. Joints represent discontinuities and may be the location of significant stress concentrations. The importance of correct detailing practice, with consideration given to the quality and conditions of site work, is emphasized.

9.2 Vertical joints

Vertical joints act primarily to transfer vertical shear forces between elements. In addition, the joint may be required to resist flexure and shear in a direction perpendicular to the plane of the walls. Factors affecting the ability of vertical joints to transfer load include:

1. Shape of the joint.
2. Reinforcement within the joint and across it.

3. Quality of concrete within the joint and in the adjacent wall panels.

4. Condition of the contact surfaces.

5. Curing conditions of the joint material.

6. Presence of reinforcing ties, within the floors, which pass across the joint.

7. Strength of mechanical connections or castellations (vertically spaced keys) within the joint.

9.3 Types of vertical joints

With respect to the behavior of precast walls subject to lateral load, vertical joints may be classified²³ as rigid, semi-rigid, or open (see Fig. 12).

9.3.1 Rigid—The shear deformation of the joint is of the same order of magnitude as the shear deformation of the panel. A rigid joint will provide for full interaction of the connected panels up to design load.

9.3.2 Semi-rigid—The shear defor-

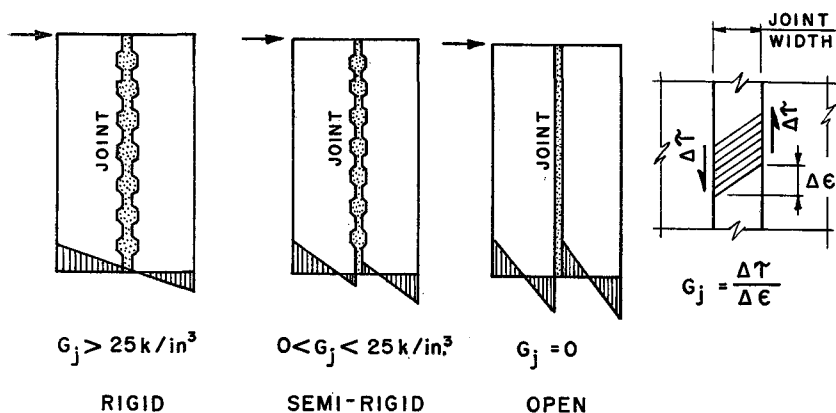


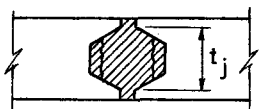
Fig. 12. Classification of vertical joints.

mation of the joint is significantly more than the shear deformation of the panel. A semi-rigid joint will generally provide full interaction until such load (less than the design load) is reached that the joint will deform at a faster rate than the adjacent panels.

9.3.3 Open—No shear forces can be transferred.

9.3.4—Representative values of the modulus of rigidity²⁴ for a specific castellated keyed joint are as follows:

Actual Stress Level, τ	Modulus of Rigidity, G_j	
	kg/cm ³	kips/in. ³
$0.9\tau_u$	100	3.6
$0.8\tau_u$	200	7.2
$0.5\tau_u$	1000	36.1
$0.3\tau_u$	5000	180.6



$$\tau = V/(t_j H) \quad (15)$$

where

V = loaded shear on vertical panel joint

H = height of panel

τ = loaded joint stress

τ_u = ultimate capacity of joint as determined by test

Tables, such as shown above, will indicate the amount of load transfer across the vertical joint.¹⁴

9.3.5—Joints constructed of cast-in-place concrete may be classified as smooth or castellated. Fig. 13 shows some representative shapes of these joints.

9.4 Vertical joint behavior

Tests indicate the following general behavior characteristics of concreted vertical joints in walls when subjected to shearing forces along the length of joint:

9.4.1—Joints without horizontal reinforcing steel across the joint will generally fail due to bond destruction and subsequent slippage at random values of shear stress.

9.4.2—Joints with horizontal reinforcing steel exhibit a considerably

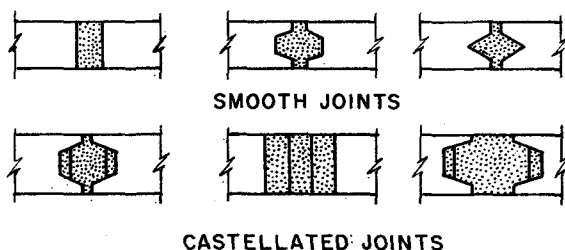


Fig. 13. Typical vertical joints.

greater load capacity, and a smaller dispersion of test results.

9.4.3—The arrangement of horizontal reinforcing steel has no significant effect on joint behavior. Reinforcement may be uniformly spaced along the length of the joint, or concentrated at discrete locations (e.g., within the floor zones).

9.4.4—Vertical reinforcing steel within the joint may contribute to strength, particularly when the horizontal reinforcement within the joint is in the form of interconnected loops, or mechanically connected.

9.5 Shear connections

Design recommendations for joints subjected to shearing forces along the length of the vertical joint are as follows:

9.5.1—Joints should be reinforced unless the adjacent panels are designed to act separately.

9.5.2—As an acceptable alternate to the use of cast-in-place reinforced concrete joints, connections may be accomplished by mechanical methods, i.e., by bolting or welding.

9.5.3—Connections (reinforced concrete or mechanical) may be designed in accordance with the shear-friction theory.

For smooth joints, the shear-fric-

tion coefficient, μ , may be taken as 0.7 when determining the reinforcement.

For castellated joints, the amount of reinforcement required will depend on the shape and spacing of the keys (Fig. 14).

The general shear-friction equation can be used:

$$A_s = \frac{V_u}{\phi f_y \mu} \quad (16)$$

where

A_s = required horizontal reinforcement

V_u = design shear on joint

$\mu = 1.4$

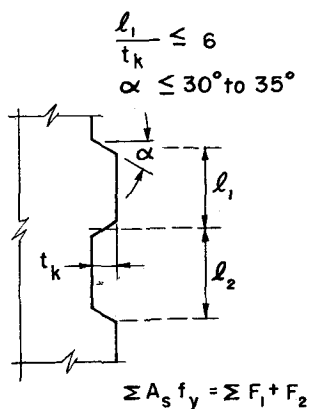
Based on the concepts of shear-friction, F_2 will become zero prior to development of the maximum value of F_1 . Therefore all the tension is generated at the key^{18,19}.

9.5.4—Where ductility is an important design consideration (e.g., in Seismic Zones 2 and 3) vertical joints should have castellations (keys) spaced the full height of the joint. Mechanical connections uniformly spaced may also be used.

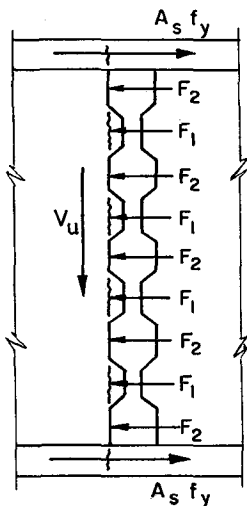
9.5.5—The contribution of the floor tie reinforcement that crosses the vertical joint may be included in the determination of total joint resistance.

LIMITING CONDITIONS:

$$\frac{l_1}{t_k} \leq 6$$

$$\alpha \leq 30^\circ \text{ to } 35^\circ$$


$$\Sigma A_s f_y = \Sigma F_1 + F_2$$



SHEARING OF KEYS

Fig. 14. Shearing of castellations in a vertical joint.

9.5.6—Consideration should be given to shrinkage and to differential shortening between adjacent panels.

9.6 Horizontal joints

Horizontal joints are a particularly critical element in the performance of large panel structures. A correctly detailed joint not only will perform as intended once the structure is complete, but it will also permit:

1. Construction to proceed with a minimum of restriction on erection methods, and a minimum of delay.
2. Visual examination of the connection as it is accomplished.
3. Ease of placement of any reinforcement and concrete within and across the joint.

Continuous chases or conduits should not be permitted within that portion of a joint that is intended to transmit vertical load.

9.7 Types of horizontal joints

General types of horizontal joints

(Fig. 15) are as follows:

1. Closed or platform joint
2. Wedge joint³²
3. Open joint

The three types of bearing conditions are as follows:

1. Concrete on concrete (partial bearing)
2. Concrete on grout (total bearing)
3. Concrete on bearing pads

9.8 Loads

Various loads which may be applied to a horizontal joint are as shown in Fig. 16. The symbols in the sketch are as follows:

- P = design loads applied from wall above
 V_w = horizontal shear from wall above
 M = end moments from floor loads
 V_f = vertical shear from floor loads

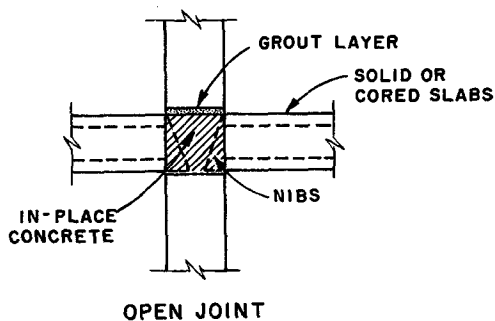
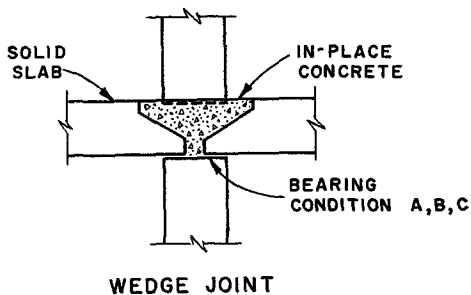
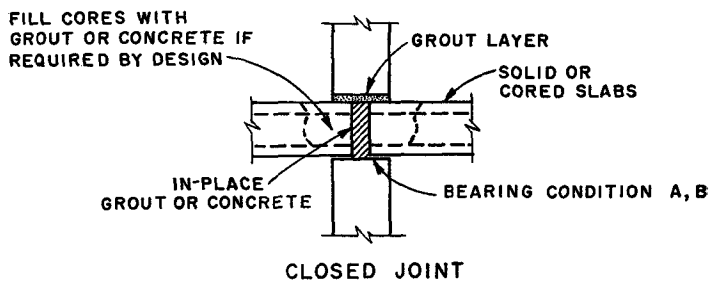


Fig. 15. Types of horizontal joints.

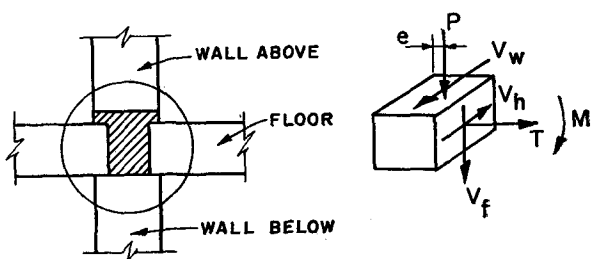


Fig. 16. Loads on horizontal joint.

V_h = horizontal shear in plane of floor due to diaphragm action

T = horizontal thrust in plane of floor (tension or compression) due to diaphragm action and restraint force due to creep, shrinkage, and thermal change

Resistance to the forces in the plane of the floor (M , T , V) is provided in the usual manner in accordance with flexure and shear theo-

ries. At any wall where the floor reinforcing steel does not continue across the joint, e.g., at an end wall, care must be taken to insure that these in-plane forces are properly transferred to the wall, or eliminated.

9.9 Vertical load capacity

Three methods for establishing the vertical load carrying capacity of horizontal joints are treated in Appendix 2.

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APPENDIX 1—DESIGN OF TRANSFER GIRDERS

A.1.1 General

Transfer girders supporting loadbearing walls and shear walls may be designed by this method¹² provided that the ratio $H/L > 2$.

The distribution of the wall loads at the level of the supporting girder will not be uniform, but will tend to concentrate near the columns since the lower parts of the wall tend to act in arch action.

Analysis indicates that the load distribution on the beam, and the magnitude of the tensile force in the beam (arch tie) are dependent primarily on

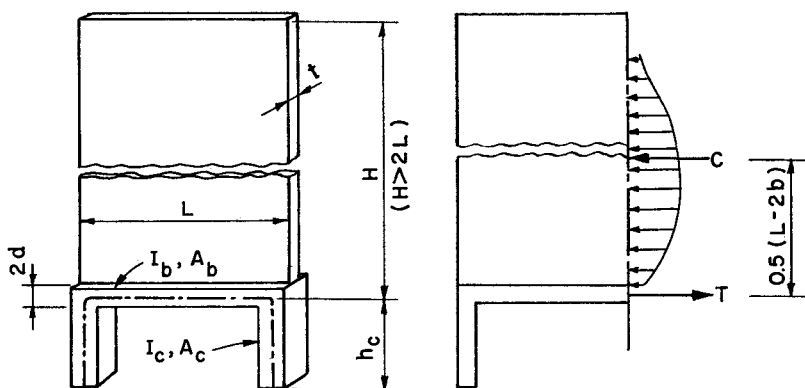
the following characteristics of the system:

1. Span and flexural stiffness of frame and wall.
 2. Effective width of column.
 3. Position and size of wall openings.
- A typical wall frame is shown in Fig. A1.

A.1.2 Tie Force

The tie force in the girder may be determined from:

$$T = W \left[\frac{1}{4} - \frac{b}{L} \left(1 - \frac{cb}{L} \right) \right] \quad (A1)$$



H = wall height

L = wall length

$2d$ = beam depth

I_b = moment of inertia of girder

A_b = cross-sectional area of girder

I_c = moment of inertia of column

A_c = cross-sectional area of column

h_c = height of column

Fig. A1. Arrangement of loadbearing wall on transfer girder.

where

T = tie force

W = total vertical load at bottom of wall

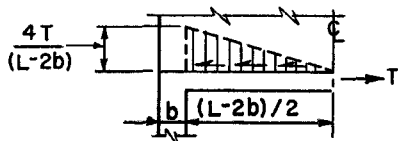
b = modified width of column, taken as 0.75 of the column width for an exterior column and 0.50 of the column width for an exterior column

c = a factor reflecting the stress distribution of the wall loads on the beam. This factor is expressed as the ratio of the (actual vertical stress of wall loads at beam support) to the (average vertical stress of wall loads).

The beam reinforcement determined from this tie force should be properly anchored at the ends of the beam.

A.1.3 Horizontal shear

The horizontal shear between base of wall and girder may be assumed to vary linearly from zero at the midspan of the girder in accordance with the sketch shown below.



Shear reinforcement should be provided between wall and girder, and may be determined in accordance with the shear-friction theory.

A.1.4 Vertical shear

The maximum vertical shear in the girder may be determined from:

$$V = (W/2) [1 - (cb/L)] \quad (A2)$$

A.1.5 Bending moments

The bending moments in the girder at midspan and at the support may be determined from:

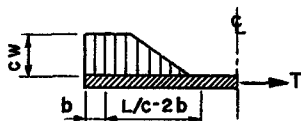
$$M_{sup} = \frac{wL}{12} \left[c \left(\frac{f}{L} \right)^2 \left(2 - \frac{f/L}{1 - 2b/L} \right) + \frac{5c''}{8} \left(1 - \frac{2b}{L} \right)^2 \right] + \frac{2Td}{3} \quad (A3)$$

$$M_{midspan} = \frac{wL}{12} \left[\frac{c(f/L)^3}{1 - 2b/L} + \frac{3c''}{8} \left(1 - \frac{2b}{L} \right)^2 \right] - \frac{Td}{3} \quad (A4)$$

where w is the average vertical stress of wall loads.

$$\text{If } \left(\frac{1}{c} - \frac{2b}{L} \right) \leq \left(\frac{1}{2} - \frac{b}{L} \right)$$

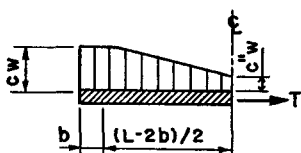
then $c'' = 0$ and $f/L = 1/c - 2b/L$.



$$\text{If } \left(\frac{1}{c} - \frac{2b}{L} \right) > \left(\frac{1}{2} - \frac{b}{L} \right)$$

$$c'' = \frac{2 - c(1 + 2b/L)}{1 - 2b/L}$$

and $f/L = \frac{1}{2} - b/L$.



The value, c , can be taken from Fig. A2, where the abscissa gives the range of the relative stiffness parameter, C_1 , on a logarithmic scale. Note that:

$$C_1 = [(L - 2b)^3 t E_w] / (I_b E_b)$$

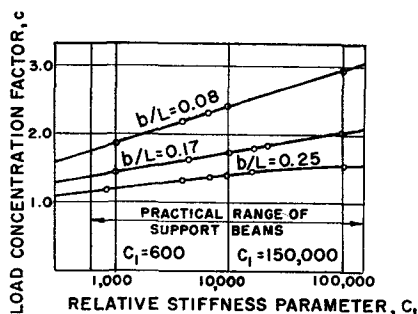
where E_w and E_b are the modulus of elasticity of the wall and transfer girder, respectively.

A.1.6 Other loads

In addition to the above, the girder

frame should be designed for any floor loads directly applied, for wind loads, for seismic loads when applicable, and for erection loads applied prior to effective composite action between wall and girder-frame.

Fig. A2. Variation of load concentration factor, c .



APPENDIX 2—LOAD CAPACITY OF HORIZONTAL JOINTS

A.2.1 General

A comprehensive design approach, including the effect of the variables discussed in Section 9.8, has not as yet been developed for all the different joint types. However, several methods for determining the load capacity of horizontal joints in large panel structures have been proposed. This Appendix presents three design procedures that apply to one common type of joint.

A.2.2 Method 1—Based on strengths of joint components

In this method (see Reference 33), the applied design load on the joint is distributed to the various components of the joint in accordance with the stress-deformation and strength characteristics of the components. Then the sum of the strengths of the components is compared to the strength of the wall as determined by ACI 318-71, Chapter 14. A "confinement factor" is permitted on the cylinder strength of the joint concrete, reflecting the confining nature of the joint.

This method applies particularly to a typical detail (see Fig. A3), where prestressed hollow-core slabs are continuously supported and extend a minimum of 2½ in. over the bearing walls. The

space between ends of slabs is filled with concrete, not necessarily of the same strength as the walls. The leveling joint between tops of slabs and bottom of wall above is solidly packed with grout of equal or greater strength than the joint concrete.

Stress transfer is complex at any joint; therefore it is necessary to clarify and isolate forces into manageable units. The floor slabs rest on bearing pads of dissimilar, "softer materials." Thus stresses can be calculated on the basis of stress-deformation characteristics of bearing pads and joint concrete.

Stress-deformation relations are shown in Fig. A4 and the ratio of elastic moduli is given as:

$$\alpha = \left[\frac{P_{n+1} - P_g}{P_g} \right] \left[\frac{t_g \Delta_g}{2t_p \Delta_p} \right] \quad (A5)$$

Average values for α for specific joint materials are given in Table A1. From this table:

$$P_g = \frac{P_{n+1}}{1 + 2\alpha(t_p/t_g)} \quad (A6)$$

The corresponding stress in the grout is:

$$f_c(\text{grout}) = P_g / (12t_g) \leq \beta f'_c \quad (A7)$$

The factor, β , considers the confinement of the joint concrete and

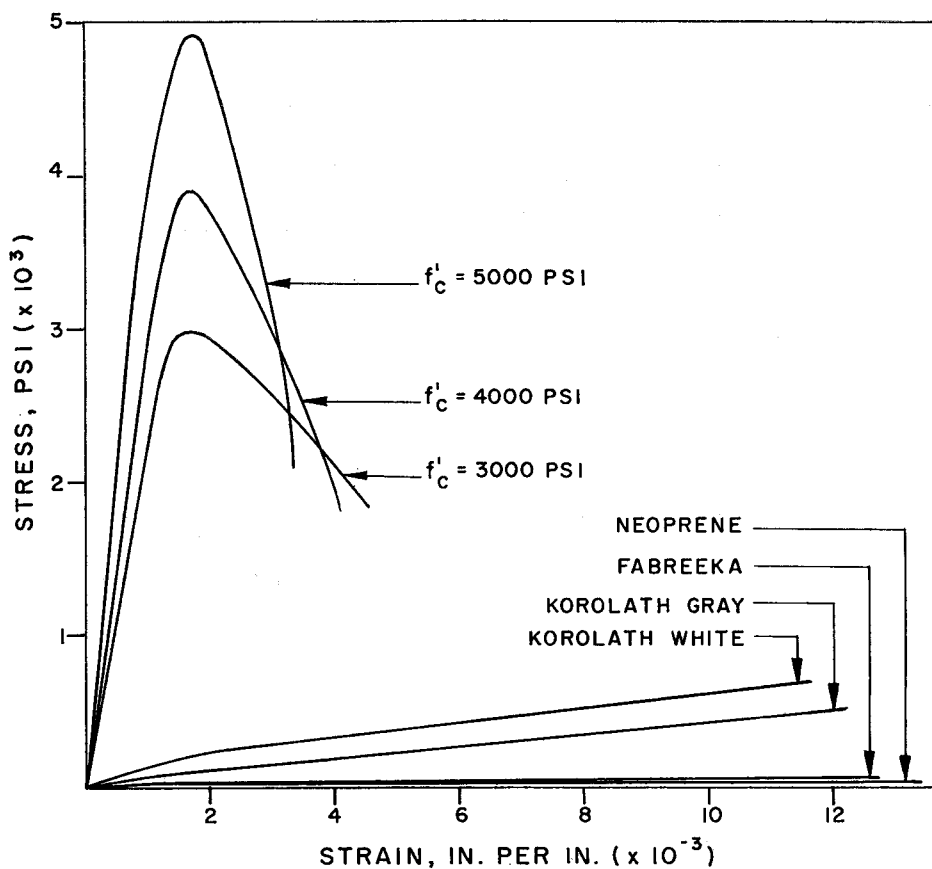


Fig. A4. Strength-deformation characteristic for grout and common bearing pads (see Reference 33).

Table A1. Average values for some common bearing pad materials (see Reference 33).

Type	f'_c (Grout)		
	3000 psi	4000 psi	5000 psi
Korolath White	0.0624	0.0541	0.0484
Korolath Gray	0.0544	0.0472	0.0422
Neoprene 50	0.0042	0.0036	0.0033
Neoprene 70	0.0063	0.0055	0.0049
Fabreeka	0.0029	0.0025	0.0023

Note: Neoprene - values based on shape factor of 7, for $f_p \leq 500$ psi.

Fabreeka - values based on $f_p \leq 500$ psi.

Example: Determine forces and stresses in horizontal joint (see Fig. A5)

Typical apartment house structure

Number of stories: 18

Live load reduction factor considered.

Spacing of bearing walls: 36 ft center to center.

Width of walls: 8 in.

Bearing pads: 2 in. wide, 1/8 in. thick, gray Korolath.

Loads: live load = 40 psf (use 50 percent reduction)

Partitions plus misc. = 20 psf

Floor slabs = 60 psf

Dead load = 80 psf

$$w_u = 1.4 \times 0.08 + 1.7(1 - 0.5) 0.04 = 0.146 \text{ ksf}$$

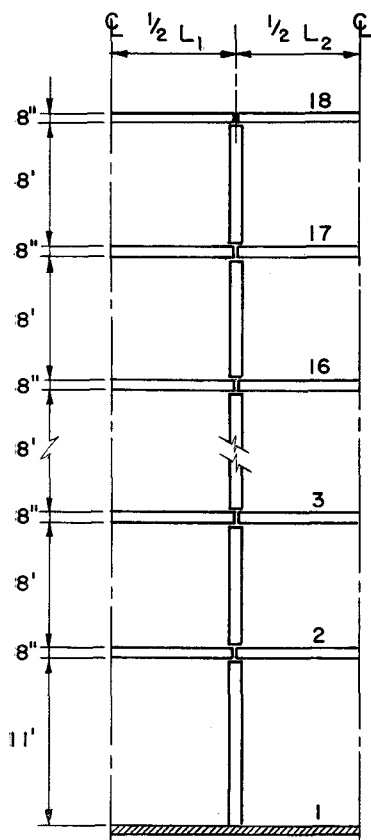


Fig. A5. Typical interior bay.

$$P_{u2} = [18 - 1] [(36 \times 0.146) + (1.4 \times 0.15 \times 8 \times 0.66)] = 108.4 \text{ kips per ft}$$

$$P_{u1} = 108.4 + (36 \times 0.146) + (1.4 \times 0.15 \times 11 \times 0.66) = 116 \text{ kips per ft}$$

Check concrete strength required for walls on first and second floors:

First floor

$$f'_c = P_{u1}/c_w = 116/30.6 = 3800 \text{ psi}$$

Second floor

$$f'_c = P_{u2}/c_w = 108.4/33.6 = 3300 \text{ psi}$$

Therefore, use an $f'_c = 4000 \text{ psi}$.

Check the grout capacity and strength at Level 2:

Use $f'_c = 3000 \text{ psi}$; $\alpha = 0.0544$

$$P_g = \frac{108.4}{1 + 2 \times 0.0544(2/2)} = 97.8 \text{ kips per ft}$$

$$f_c(\text{grout}) = 97,800/(12 \times 2) = 4075 \text{ psi} (< 2 f'_c)$$

Check bearing pad stress:

$$P_{pl} = P_{pr} = (108.4 - 97.8)/2 + (36/2) 0.146 = 7.93 \text{ kips per ft}$$

$$f_c(\text{pad}) = \frac{7930}{0.667 \times 2 \times 12} = 534 \text{ psi}$$

Note that this stress is less than the manufacturer's recommended ultimate stress.

Check drypack stress:

$$f_c(\text{drypack}) = 108.4/(7.14 \times 7) = 2400 \text{ psi}$$

Use $f'_c = 3000 \text{ psi}$.

It should be noted from the example that, in addition to the floor loads of the level under consideration, only minor forces from the wall above are transferred through the bearing pads.

This particular feature reinforces the validity of the assumptions made for the analysis of the structure according to the "hinged joint" pattern and literally recenters loads at each floor level.

Table A2. Factors c_w for calculating bearing capacity of concrete walls (ACI 318-71)

$P_u = (c_w)(f'_c) \text{ ksi}$

Height h_w , ft	Thickness, t , in.								
	12	11	10	9	8	7	6	5	4
7	53.7	48.9	44.0	39.3	34.4	29.4	24.3	19.0	13.3
7.5	53.5	48.7	43.7	39.0	34.0	29.0	23.8	18.4	12.6
8	53.2	48.4	43.4	38.6	33.6	28.5	23.3	17.8	11.8
8.5	52.9	48.0	43.0	38.2	33.2	28.0	22.7	17.1	10.9
9	52.6	47.7	42.7	37.8	32.7	27.5	22.1	16.3	10.0
9.5	52.3	47.4	42.3	37.4	32.2	27.0	21.4	15.6	9.1
10	51.9	47.0	41.9	36.9	31.7	26.4	20.8	14.8	
10.5	51.6	46.6	41.5	36.5	31.2	25.8	20.0	13.9	
11	51.2	46.2	41.0	36.0	30.6	25.1	19.3	13.0	
12	50.4	45.4	40.1	34.9	29.4	23.8	17.7	11.1	
13	49.5	44.4	39.0	33.8	28.2	22.3	16.0		
14	48.6	43.4	37.9	32.5	26.7	20.7	14.1		
15	47.6	42.3	36.7	31.2	25.2	18.9			
16	46.5	41.1	35.4	29.7	23.6	17.1			
17	45.4	39.9	34.1	28.2	21.9	15.2			
18	44.2	38.5	32.6	26.6	20.1				
19	42.9	37.1	31.1	24.9	18.2				
20	41.6								

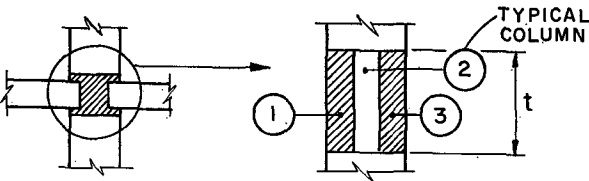
Note: c_w factors below double line apply to precast walls only; all other factors applicable to all concrete walls.

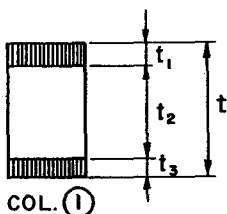
A.2.3 Method 2—Elastic analysis of joint

This method (see Reference 4) assumes that the elastic response of each component of the joint is known. It will permit a determination of the stress distribu-

tion, under service load conditions, on the various components of the joint.

The joint is divided into a series of discrete vertical “columns” as shown in the sketch. The amount of load that each “column” supports will be a func-





tion of the stiffness (spring constant) of that particular "column." Each of the layers of the "columns" may have a different thickness, and may be of different materials. The equivalent spring constant, K , of any "column" may be determined by:

$$K = \frac{t}{\sum (t_i/E_i)} \quad (A10)$$

where

t = total height of column

t_i = thickness of an individual layer

E_i = modulus of elasticity of that layer

The vertical stress to which each "column" of the floor joint is subjected can be determined, once the spring constant of each "column" has been established by:

$$f_i = (P_e/S) (K_i/K) \quad (A11)$$

where

f_i = stress on a particular "column"

P = total vertical service load applied to the joint

e = eccentricity of the force P , measured from the same axis as used to determine S

S = statical moment of a part of the reduced cross section loaded with an equivalent rectangular stress distribution, calculated in reference to one of the edges of the joint.

K_i = spring constant of the particular "column" being investigated

K = spring constant of the reference "column"

Example: Determine distribution of stress in a horizontal joint (see sketch below)

Step 1—Determine the spring constant for each "column"

Column 1:

$$K_1 = \frac{10.5}{\frac{1.5}{2000} + \frac{8}{4000} + \frac{1.0}{2000}} = 3231$$

$$K_1/K = 3231/3231 = 1.00$$

Column 2:

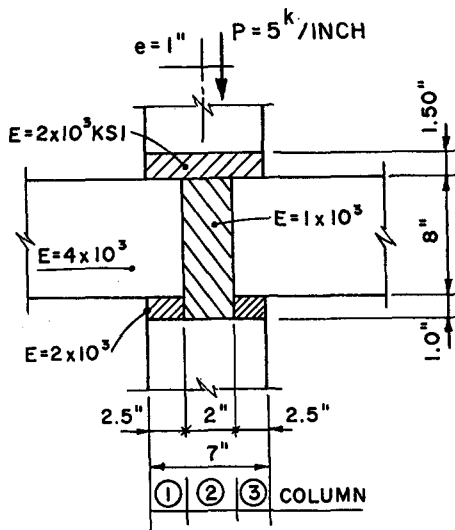
$$K_2 = \frac{10.5}{\frac{1.5}{2000} + \frac{9.0}{1000}} = 1077$$

$$K_2/K = 1077/3231 = 0.333$$

Column 3:

$$K_3 = \frac{10.5}{\frac{1.5}{2000} + \frac{8}{4000} + \frac{1.0}{2000}} = 3231$$

$$K_3/K = 3231/3231 = 1.00$$



Design example

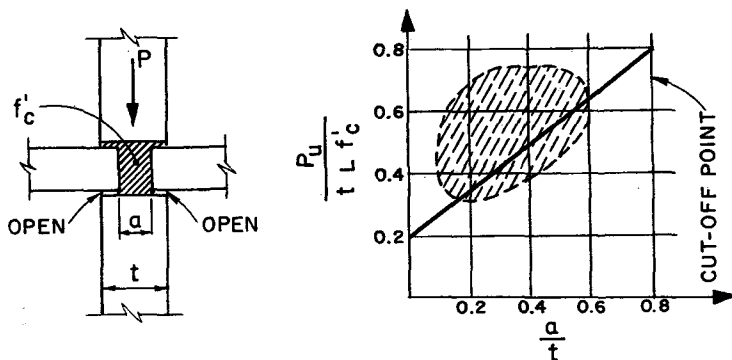
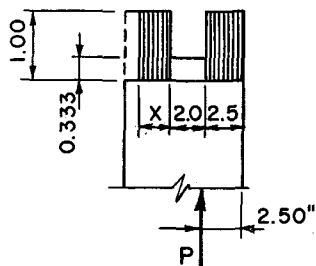


Fig. A6. Test results on a typical horizontal joint.

Step 2—Determine the dimensions of the equivalent stress blocks (see sketch)



$$1.00 \times 2.50 = 2.500 \times 1.25 = 3.125$$

$$0.333 \times 2.00 = 0.666 \times 3.50 = 2.331$$

$$1.00(x) = (x)(4.5 + x/2) =$$

$$4.5x + x^2/2$$

The above equations reduce to:

$$2.50(x + 3.166) = 5.456 +$$

$$4.5x + x^2/2 \text{ from which } x = 0.986 \text{ in.}$$

Step 3—Determine S

$$S = x^2/2 + 4.5x + 5.456$$

$$S = (0.986)^2/2 + 4.5 \times 0.986 + 5.456$$

$$= 10.379$$

Step 4—Determine stresses in each "column"

Column 1:

$$f = [(5 \times 2.5)/10.379] [(1/1)]$$

$$= 1204 \text{ psi}$$

Column 2:

$$f = [(5 \times 2.5)/10.379] [(0.333/1)]$$

$$= 400 \text{ psi}$$

Column 3:

$$f = [(5 \times 2.5)/10.379] [(1/1)]$$

$$= 1204 \text{ psi}$$

Check

$$\Sigma V = (1.204 \times 2.5) + (0.400 \times 2) +$$

$$(1.204 \times 0.986)$$

$$= 5.0 \text{ kips (ok)}$$

A.2.4 Method 3—Tests on a typical horizontal joint

Tests by the Danish Structural Research Center, and by others, for the type of joint shown in Fig. A6, have been plotted, and the results generally fall within the shaded zone of the curve as indicated. Provided that the strength of the grout within the joint is about the same as the strength of the concrete in the wall, the following empirical relation is suggested for determining the design strength of the joint:

$$P_u = t L f'_c [0.2 + 0.75(a/t)] \quad (A12)$$

where

a = width of grout bed

f'_c = compressive strength of grout

L = length of wall

P_u = design strength of the joint per unit length

Expressions such as Eq. (A12), based on tests applied to one type of joint, may not be applicable to other joint types or different materials or loading conditions.

The Behavior of Reinforced Concrete Corbels

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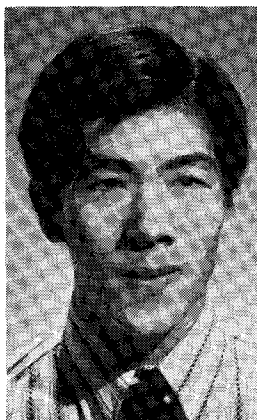
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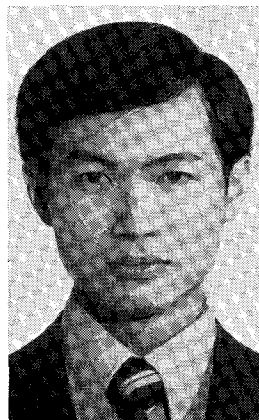
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Review of current design provisions for corbels

Corbels (or brackets) projecting from the faces of reinforced concrete columns are used extensively in precast concrete construction to support primary beams and girders.

The design of corbels is governed by the provisions of Section 11.14 of ACI 318-71.¹ Under these provisions, corbel design may either be based on the rather complicated empirical Eqs. (11-28) and (11-29), which are valid for $a/d \leq 1.0$, or if a/d is limited to 0.5, a simpler method of design based on the shear-friction provisions of Section 11.15 may be used.

However, when the shear-friction provisions are used for corbel design, the limitations on quantity and spacing of reinforcement specified in Section 11.14 must still be observed. These limitations derive from the recommendations of Kriz and Rath.²

ACI 318-71 requires that unless special measures are taken to avoid horizontal tension forces, all corbels shall be designed for an appropriate horizontal force N_u in addition to the vertical load V_u , and that N_u/V_u shall not be taken as less than 0.2.

In this case the reinforcement ratio ρ is limited by Section 11.14 to

$$\text{max. } \rho = A_s/bd = 0.13f'_c/f_y$$

that is:

$$\text{max. } \rho f_y = 0.13f'_c$$

Therefore, using shear-friction:

$$\text{max. } v_u = \mu(0.13f'_c + \sigma_{Nx})$$

The normal stress σ_{Nx} is negative for tension. Therefore, when a tension force N_u acts:

$$\sigma_{Nx} = -\frac{N_u}{V_u}(v_u)$$

that is:

$$\text{max. } v_u = \mu \left[0.13f'_c - \frac{N_u}{V_u}(\text{max. } v_u) \right]$$

$$\text{max. } v_u \left(\frac{1}{\mu} + \frac{N_u}{V_u} \right) = 0.13f'_c$$

Synopsis

An experimental study is reported of the behavior of reinforced concrete corbels subjected to both vertical and horizontal loads.

Twenty-eight corbel specimens were tested, of which twenty-six contained horizontal stirrup reinforcement.

The variables included in the study were the shear span to effective depth ratio, the ratio of the vertical load to the horizontal load, the amounts of main tensile and stirrup reinforcement, and the type of aggregate.

Criteria for the design of horizontal stirrup reinforcement are developed.

It is shown that, subject to provision of the recommended amount of stirrup reinforcement, the useful ultimate strength of corbels can be taken to be the lesser of

(a) the shear strength of the corbel-column interface, calculated using either the shear-friction provisions of Section 11.15 of ACI 318-71 or the modified shear-friction equation, and;

(b) the vertical load corresponding to the development of the flexural ultimate strength of the corbel-column interface.

In the next issue a follow-up paper will propose specific code provisions for designing corbels together with numerical examples.

therefore:

$$\max. v_u = \frac{0.13f'_c}{\frac{1}{\mu} + \frac{N_u}{V_u}}$$

For corbels monolithic with their supporting column, for which $\mu = 1.4$, the maximum allowable shear stress due to the limitation ρ not greater than $0.13f'_c$ will vary from $0.142f'_c$ when N_u/V_u has its minimum value of 0.2, to $0.076f'_c$ when N_u/V_u is equal to unity,* i.e., v_u (max.) according to Section 11.14 varies from 71 percent down to 38 percent of the v_u (max.) allowed under the shear-friction provisions of Section 11.15.

The requirement of Section 11.14 that horizontal stirrups or ties with an area A_h not less than $0.50A_s$ be provided, is designed to prevent a premature diagonal tension or "diagonal splitting" failure of the corbel. This requirement is also based on Kriz and Rath's² recommendation.

The provision leads to heavy stirrup requirements when a large N_u acts with V_u . However, the data obtained from tests of corbels with stirrups subject to combined loading, on which Kriz and Rath's based their recommendation, was rather limited, involving only five specimens with arbitrarily proportioned stirrup reinforcement.

In addition to the quantity of stirrups provided, the following were also varied in these five specimens—the shear span to depth ratio, the effective depth, the N_u/V_u ratio, and the concrete strength. (The reinforcement ratio $\rho = A_s/bd$ was the same in all the specimens with stirrups.)

*No upper limit to N_u/V_u is specified in Section 11.14 of ACI 318-71, but Eq. (11-28) in Section 11.14 does not yield sensible results if a value greater than 1.0 is substituted for N_u/V_u in the equation.

†The reinforcement ratio ρ used in Eq. (11-28) is based on the area of the main tension reinforcement only ($\rho = A_s/bd$).

The results obtained in these five tests were also the basis for a conclusion that no consistent increase in shear capacity was obtained in a corbel subject to combined loading, when stirrups were provided. The requirement in Section 11.14 of ACI 318-71, that the main tension reinforcement only be considered† when calculating the strength of a corbel subject to combined loading, stems from this conclusion of Kriz and Rath's.

Purpose of this study

The study reported in this paper was directed toward the extension of the shear-friction method of corbel design to corbels having shear span to depth ratios a/d of up to 1.0, subject to combinations of vertical and horizontal loads such that $N_u/V_u \leq 1.0$.

It has already been shown³ that the simultaneous action of a moment equal to the flexural ultimate strength of a cracked section does not reduce the shear which can be transferred across the crack, and that the arbitrary limitation of the use of the shear-friction design method to cases when $a/d \leq 0.5$ is unwarranted.

On the basis of the work reported earlier,³ it was tentatively proposed that, providing a premature diagonal tension failure of the corbel is prevented by the provision of an appropriate minimum amount of stirrup reinforcement, the ultimate strength of a corbel can be calculated by taking it to be the lesser of

(a) The shear strength of the corbel interface, calculated using the shear-friction provisions of Section 11.15 of ACI 318-71; and

(b) The vertical load corresponding to the development of the flexural ultimate strength of the corbel-column interface, using the provisions of Section 10.2 of ACI 318-71.

The experimental study reported here

was designed to check the range of applicability of this approach to corbel design and to develop rational requirements for stirrups in corbels.

Preliminary considerations

Consider the corbel shown in Fig. 1a, acted on by a shear V_u and a horizontal force N_u . It is assumed that the main tension reinforcement A_s and the horizontal stirrups A_h can develop their yield strengths, f_y and f_{vy} , respectively.

The maximum values of the reactive forces acting at the interface between the corbel and the column will then be as shown in Fig. 1b, if shear resistance along the interface is calculated using the shear-friction hypothesis.

For the corbel to remain in equilibrium the following requirements must be satisfied:

$$1. \sum F_y = 0$$

$$\text{i.e., } V_u \leq \mu C \quad (1)$$

$$2. \sum F_x = 0$$

$$\text{i.e., } N_u \leq \phi A_s f_y + \phi A_h f_{vy} - C \quad (2)$$

$$3. \sum M = 0$$

$$\text{i.e., } V_u a + N_u (h - d + jd) \leq \phi A_s f_y jd + \phi A_h f_{vy} j_1 d \quad (3)$$

Now

$$jd = d - \frac{1}{2} \left(\frac{\phi A_s f_y + \phi A_h f_{vy} - N_u}{0.85 f'_c b} \right)$$

$$= j'd - \frac{1}{2} \left(\frac{\phi A_h f_{vy}}{0.85 f'_c b} \right)$$

where $j'd$ would be the internal lever arm if only flexural reinforcement of strength $(\phi A_s f_y - N_u)$ was acting in the section, that is:

$$j'd = d - \frac{1}{2} \left(\frac{A_s f_y - N_u}{0.85 f'_c b} \right)$$

Substituting this value for jd in Eq. (3) and simplifying, we obtain:

$$V_u a + N_u (h - d + jd) \leq \phi A_s f_y j'd + \phi A_h f_{vy} \left\{ j_1 d - \frac{1}{2} \left(\frac{0.85 f'_c b}{\phi A_s f_y} \right) \right\}$$

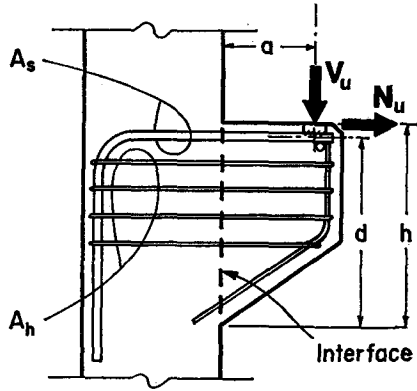


Fig. 1a. A typical corbel.

We may conservatively neglect the contribution of the stirrup force $\phi A_h f_{vy}$ to the resistance moment and write:

$$V_u a + N_u (h - d + jd) \leq \phi A_s f_y j'd \quad (4)$$

Therefore:

$$A_s \geq \frac{V_u a + N_u (h - d)}{\phi f_y j'd} + \frac{N_u jd}{\phi f_y j'd}$$

Now $jd/j'd$ is slightly less than 1.0. Therefore, it is conservative to write:

$$A_s \geq \frac{V_u a + N_u (h - d)}{\phi f_y j'd} + \frac{N_u}{\phi f_y} \quad (5)$$

That is:

$$A_s \geq A_f + A_t \quad (6)$$

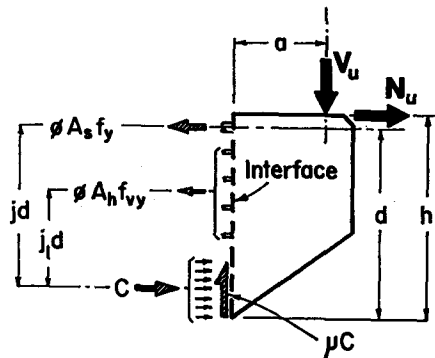


Fig. 1b. The corbel as a "free body."

where

A_f = area of reinforcement necessary to resist the applied moment $[V_u a + N_u(h - d)]$

A_t = area of reinforcement necessary to resist the horizontal force N_u

Eliminating C between Eqs. (1) and (2), and simplifying, we obtain:

$$A_h \cong \frac{1}{f_{vy}} \left(\frac{V_u}{\phi \mu} + \frac{N_u}{\phi} - A_s f_y \right) \quad (7)$$

If $f_y = f_{vy}$, we may write:

$$A_h \cong \frac{V_u}{\phi \mu f_y} + \frac{N_u}{\phi f_y} - A_s$$

or

$$A_h \cong A_{vf} + A_t - A_s \quad (8)$$

where A_{vf} is the area of reinforcement necessary to carry the shear V_u calculated using the shear-friction Eq. (11-30) of ACI 318-71.

Substituting for A_s in Eq. (8) from Eq (6) we have:

$$A_h \cong A_{vf} - A_f \quad (9)$$

It is possible that Eq. (9) could yield a very small or zero value for A_h . However, this would not be admissible, since it is presupposed that the amount of horizontal stirrups present in the corbel is sufficient to prevent a premature diagonal tension failure of the corbel. A suitable minimum amount of stirrups must therefore be specified to ensure this.

In this study, it was decided to start with the hypothesis that the minimum amount of stirrups should be that required to carry the difference in shear between the ultimate shear the corbel was to carry, and the shear at which it would fail in diagonal tension if no stirrups were provided. It was therefore necessary to determine a suitable way of calculating the shear strength of a corbel without stirrups.

After reviewing the literature, it was decided to examine the applicability to corbels of the following equation devel-

oped by Zsutty⁴ for the shear strength of directly loaded reinforced concrete beams without stirrups and having shear span to depth ratios of 2.5 or less:

$$v_{u2} = 60 \left(f_c \rho \frac{d}{a} \right)^{1/3} \left(\frac{2.5}{a/d} \right) \quad (10)$$

Eq. (10) was developed from the following equation for the shear strength of slender beams ($a/d > 2.5$), previously proposed by Zsutty:⁵

$$v_{u1} = 60 \left(f_c \rho \frac{d}{a} \right)^{1/3} \quad (11)$$

A study was made of the applicability of Eq. (10) to those vertically loaded corbels tested by Kriz and Rath² which were reported to have failed by "diagonal splitting" (i.e., diagonal tension).

The initial calculations showed that Eq. (10) grossly overestimates the shear at diagonal tension failure in members with such small values of a/d . A direct comparison was therefore made of the measured shear stresses at "diagonal splitting" failure, v_u , and the "slender beam" shear strengths, v_{u1} , calculated using Eq. (11).

This is shown in Fig. 2. By so doing, it was hoped to determine whether Zsutty's "arching factor," $2.5/(a/d)$, in Eq. (10) could be modified in a simple manner, or limits be placed on its applicability, so that Eq. (10) could be made applicable to corbels with small a/d values.

It can be seen that reasonable agreement can be obtained between measured and calculated stresses if an upper limit of 2.5 is placed upon the factor $[2.5/(a/d)]$.

The upper limit controls when a/d is 1.0 or less.

It was therefore decided that in the first corbel specimens to be tested in this study, the minimum horizontal web reinforcement would be made equal to

$$\rho_h f_{vy} (\text{min.}) = v_u - v_{u2} \quad (12)$$

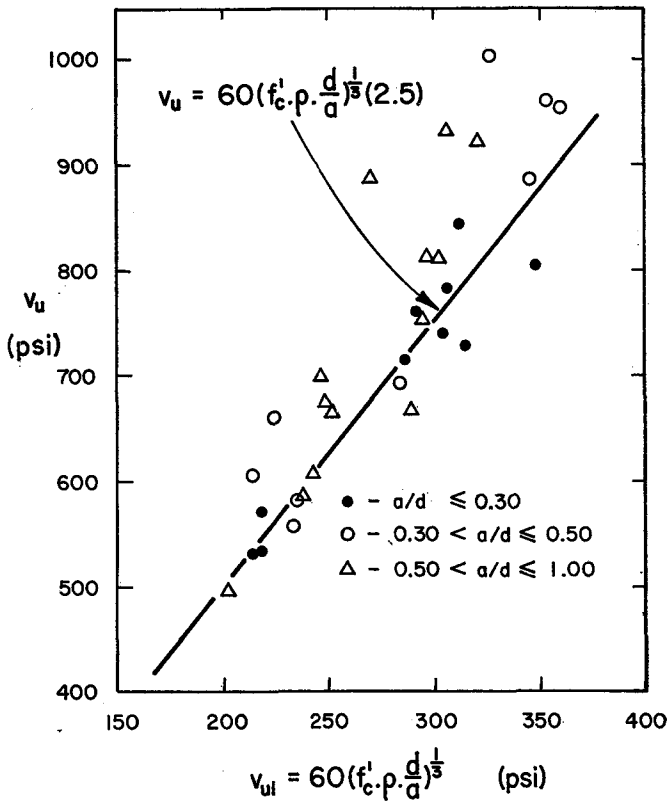


Fig. 2. Comparison of measured and calculated shear stresses at "diagonal splitting" failure in tests of corbels by Kriz and Rath² (Corbels subject to vertical load only.)

where

$$\rho_h = A_h/bd$$

v_u = design ultimate shear stress

$$v_{u2} = 60 \left(f'_c \rho \frac{d}{a} \right)^{1/3} K \quad (13)$$

where $K = 2.5/(a/d)$
but not greater than 2.5

For $a/d \leq 1.0$, Eq. (13) becomes:

$$v_{u2} = 150 \left(f'_c \rho \frac{d}{a} \right)^{1/3} \quad (14)$$

Hence:

$$\rho_h f_{vy} (\text{min.}) = v_u - 150 \left(f'_c \rho \frac{d}{a} \right)^{1/3} \quad (15)$$

Experimental Study

A total of 28 corbel specimens were tested, 26 of which contained horizontal stirrup reinforcement. The test program was developed as it progressed and is summarized in Table 1.

Series A and B were tested to study minimum stirrup requirements in corbels designed to carry vertical loads only. Series C, D and E were tested to check the design method proposed when applied to corbels made of sand and gravel concrete, subject to both vertical and horizontal loads. In Series C and E, the design ultimate shear

Table 1. Summary of test program.

Series	No. of Specimens	Type of Concrete	Horizontal Stirrups	Design N_u/V_u	Design v_u (psi)
A	2	Sand & Gravel	No	0	800
B	4	Sand & Gravel	Yes	0	800
C	4	Sand & Gravel	Yes	0.75	800
D	3	Sand & Gravel	Yes	1.0	565
E	4	Sand & Gravel	Yes	1.0	800
F	4	All-lightweight	Yes	1.0	800
G	1	All-lightweight	Yes	0	800
H	5	Sand & Gravel	Yes	1.0	1200
J	1	All-lightweight	Yes	1.0	560

stress was made equal to $0.2f'_c = 800$ psi the maximum allowable according to the shear-friction provisions of Section 11.15 of ACI 318. In Series D and E, the design N_u/V_u was made equal to the proposed maximum value of 1.0 for this method of design. Within each of these tests series, a/d was varied, having values up to and including 1.0.

Series F, G and J were tested to check the design method proposed when applied to corbels made of all-lightweight concrete, subject to both vertical and horizontal loads. In these series also, the limiting values of v_u , N_u/V_u , and a/d were explored.

Series H was tested to examine the possibility of using the following previously proposed⁷ equation for the shear transfer strength of sand and gravel concrete, in place of the shear-friction equation when designing corbels as proposed.

$$v_u = 0.8 \rho f_y + 400 \text{ psi} \quad (16)$$

but not greater than $0.3f'_c$.

The specimens were designed for v_u equal to the maximum value of $0.3f'_c$ simultaneously with N_u/V_u having its maximum value of 1.0 and a/d having values up to and including 0.68.

Grade 60 reinforcement was used in Series E through J, to ensure that any

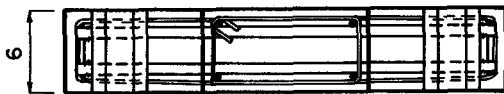
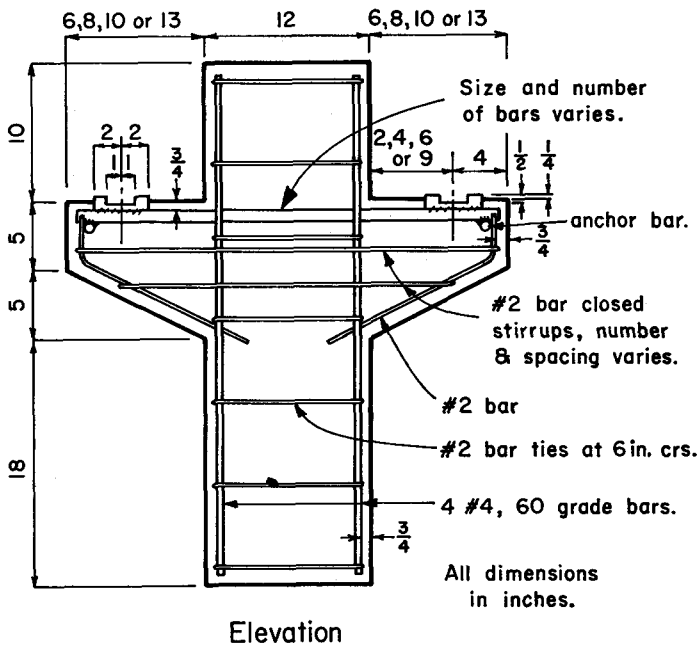
conclusions reached would be valid for corbels reinforced with this higher strength steel.

Test specimens

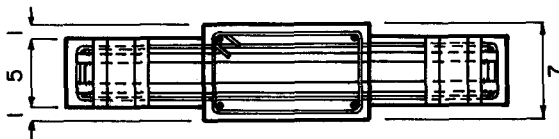
Each specimen consisted of a 38-in. length of column with two corbels projecting from the column in a symmetrical fashion, as shown in Fig. 3. The main tension reinforcement consisted of parallel straight bars, anchored by short bars of equal diameter welded across their ends. The closed horizontal stirrups were #2 deformed bars and were uniformly distributed in the two-thirds of the effective depth nearest the main tension reinforcement.

Steel bearing plates (4 x 1 x 6 in.) were welded to the main tension reinforcement. Series A, B, and G specimens had plain bearing plates. The remainder of the specimens had grooved bearing plates, as shown in Fig. 3, to facilitate applying the horizontal force to the corbels.

The amount and strength of reinforcement used in each specimen are shown in Tables 2 and 3. The design compressive strength of the concrete at test was 4000 psi for all specimens. The actual concrete strengths at test are given in Tables 2 and 3.



Plan, Series A,B,C,D



Plan, Series E,F,G,H,J

Fig. 3. Typical corbel test specimens.

Design of test specimens

The test specimens were designed as proposed above, except that in the case of Series C and D, the term $N_u(h-d)$ was omitted when calculating A_s using Eq. (5). This was done because it was initially assumed that the unconservatism of this action would be cancelled by the conservatism of ignoring the resistance moment contribution of the

horizontal stirrups. However, when the test results from Series C and D were studied, this assumption was found to be incorrect. The term $N_u(h-d)$ was therefore included when calculating A_s in the design of the specimens of Series E through J. The capacity reduction factor was taken equal to 1.0 in these design calculations.

Series A—The specimens of Series A

were intended to provide a reference as to the effectiveness of the horizontal stirrups provided in the specimens of Series B. The specimens of Series A were therefore made as nearly as possible identical to the corresponding specimens of Series B, except that the horizontal stirrups were omitted.

Series B—All the specimens of Series B were designed to have an ultimate shear stress of $0.2f'_c$, i.e., 800 psi. The first three specimens of the series were provided with minimum stirrup reinforcement as proposed above, that is:

$$\rho_h f_{vy} (\text{min.}) = v_u - 150 \left(f'_c \rho \frac{d}{a} \right)^{1/3} \quad (15)$$

The value of $\rho_h f_{vy} (\text{min.})$ calculated using Eq. (15) was found to be almost

a constant quantity, being 186, 183, and 179 psi for Specimens B1, B2, and B3, respectively. This is very nearly equal to $\frac{1}{3}(A_v f_{vy}/bd) = 190$ psi in this case where $v_u = 800$ psi.

This is apparently due to the occurrence of the product $(\rho \cdot d/a)$ in the equation for v_{u2} . When the shear span "a" is increased, then for a constant V_u , M_u increases and " ρ " increases almost in proportion to the increases in "a." Hence the product $(\rho \cdot d/a)$ remains approximately constant and therefore v_{u2} remains approximately constant.

For smaller values of v_u , ρ would decrease in proportion to v_u but v_{u2} would decrease less rapidly because it is proportional to $\rho^{1/3}$. The quantity $(v_u - v_{u2})$ would therefore decrease more rap-

Table 2. Details of corbel specimens (Series A, B, C & D).

Spec. No.	a/d	Main Reinforcement			Stirrups		$\rho = \frac{A_s}{bd}$	Concrete Strength f'_c (psi)
		Bars	A_s (in. ²)	f_y (ksi)	A_h (in. ²)	f_{vy} (ksi)		
A2	0.67	2 #5	0.62	46.6	0	-	0.0116	3675
A3	1.01	2 #6 +1 #3	0.99	54.1 53.6	0	-	0.0186	3850
B1	0.44	2 #4	0.40	48.5	0.20	65.0	0.0075	3630
B2	0.67	2 #5	0.62	46.5	0.20	67.0	0.0116	3450
B3	1.01	2 #6 +1 #3	0.99	54.1 53.2	0.20	64.0	0.0186	3760
B3A	1.01	2 #6 +1 #3	0.99	52.5 50.9	0.40	65.5	0.0186	4165
C1	0.45	2 #6 +1 #4	1.08	50.0 48.3	0.20	60.0	0.0203	4010
C2	0.68	3 #6	1.32	50.4	0.20	67.4	0.0248	3715
C2A	0.68	2 #6 +1 #5	1.19	50.0 47.4	0.20	65.6	0.0223	3705
C3	1.02	2 #7 +1 #6	1.64	50.0 50.9	0.40	65.5	0.0310	4385
D1	0.45	2 #6	0.88	50.2	0.10	70.0	0.0165	3910
D2	0.68	2 #6 +1 #4	1.08	47.7 47.5	0.20	64.0	0.0203	3805
D3	1.01	3 #6	1.32	48.4	0.25	67.6	0.0248	3700

idly than would $(v_u/3)$ and hence, the minimum A_h by this hypothesis would be less than $A_{vf}/3$. At this point, it appeared that a simple and conservative rule for minimum stirrup reinforcement might be that it should be not less than $A_{vf}/3$.

Specimens B1 and B2 performed in a satisfactory manner, but Specimen B3 developed an ultimate strength of only 600 psi. The characteristics of the specimens as constructed were re-examined. It was found that because the actual strength of the larger diameter bars used for A_v was less than the value of 50 ksi assumed in the design and the strength of the #2 bars used for the

horizontal stirrups was more than 50 ksi, the yield strength of the stirrups provided in Specimen B2 was very close to $A_v f_y/2$.

A fourth specimen, B3A, was therefore designed in which A_v was made the same as in Specimen B3, but a larger number of stirrups was provided so that their total yield strength $A_h f_{vy}$ was as nearly as possible equal to $A_v f_y/2$. This specimen behaved in a satisfactory manner.

It therefore appeared that it might be appropriate to require that the closed stirrups or ties parallel to the main tension reinforcement shall have a total yield strength at least equal to half the

Table 3. Details of corbel specimens (Series E, F, G, H & J).

Spec. No.	a/d	Main Reinforcement		Stirrups		$\rho = \frac{A_s}{bd}$	Concrete Strength f'_c (psi)
		Bars	A_s (in. ²)	f_y (ksi)	A_h (in. ²)	f_{vy} (ksi)	
E1	0.22	1 #6 +2 #4	0.84	59.1 67.5	0.15	64.0	0.0189
E2	0.45	2 #6 +1 #2	0.93	59.7 67.0	0.15	67.0	0.0210
E3	0.68	2 #6 +1 #4	1.08	62.5 64.0	0.25	65.0	0.243
E4	1.01	3 #6	1.32	62.5	0.35	67.0	0.0297
F2	0.45	2 #6 +1 #2	0.93	62.5 65.0	0.15	68.2	0.0210
F3	0.68	2 #6 +1 #4	1.08	62.3 64.0	0.25	64.0	0.0243
F4	1.01	3 #6	1.32	63.2	0.35	68.2	0.0297
F4A	1.01	3 #6	1.32	63.4	0.35	68.0	0.0297
G4	0.99	1 #6 +2 #3	0.66	62.5 65.5	0.30	67.4	0.0147
H1	0.23	2 #6 +1 #2	0.93	64.8 68.2	0.20	64.0	0.0210
H2	0.45	2 #6 +1 #4	1.08	62.4 64.0	0.25	64.0	0.0243
H3	0.68	3 #6	1.32	63.2	0.35	67.0	0.0297
H3A	0.68	3 #6	1.32	64.1	0.35	68.6	0.0297
H3B	0.68	2 #6 +1 #6	1.32	62.5 64.8	0.35	68.0	0.0297
J4	1.01	2 #6	0.88	64.8	0.25	64.0	0.0198

yield strength of the reinforcement required for flexure, or one-third the yield strength of the reinforcement required to resist shear (calculated using the shear-friction provisions of ACI 318-71), whichever is the greater.

If the yield point of the stirrups and the main tension reinforcement is the same, the above requirement reduces to $A_h(\text{min.}) = A_f/2$ or $A_{vf}/3$, whichever is the greater.

Series C—The corbels of this series were designed to be companion specimens to Corbels B1, B2, and B3A. The Series C corbels were provided with additional main tension reinforcement, so that the strength of the main tension reinforcement in the Series C corbel was greater than that provided in its companion Series B corbel by an amount equal to the horizontal force N_u that it was proposed to apply to the corbel concurrently with the shear.

The horizontal stirrup reinforcement was made the same as that provided in the companion Series B corbels. That is, $A_h f_{vy}$ was made as nearly as possible equal to the greatest of the values given by $\frac{1}{3} A_v f_{vy}$ or $\frac{1}{2} A_f f_y$.

This was done to check whether this amount of stirrup reinforcement is adequate when the maximum allowable horizontal force acts on the corbel concurrently with the shear. These corbels behaved satisfactorily. Therefore, this amount of stirrup reinforcement was provided in all subsequent test specimens.

Series D and E—The specimens of both these series were designed to carry an ultimate horizontal force equal to the ultimate shear force. However, the corbels of Series E were made of a reduced width, so that a design ultimate shear stress of 800 psi could be attained when using available hydraulic rams to apply the horizontal force.

Series F, G, and J—The specimens of Series F were of all-lightweight con-

crete and were designed to have the same ultimate shear strength and ultimate outward horizontal force as the corresponding specimens of Series E. However, in this case the shear-friction calculation was modified as proposed previously,⁶ μ being multiplied by the coefficient 0.75 for all-lightweight concrete contained in Section 11.3.2 of ACI 318-71.

Specimen F2 ($a/d = 0.45$) behaved satisfactorily, yielding a v_u of 825 psi. However, Specimen F4 ($a/d = 1.01$), which was tested next, yielded a v_u of only 540 psi, or $0.13f'_c$. The characteristics of the specimen as constructed were re-examined, but there was no immediately apparent reason for the poor behavior. A duplicate specimen F4A was fabricated and tested. It behaved the same as Specimen F4, yielding a v_u of $0.14f'_c$. A specimen G4, with the same a/d as Specimen F4, was designed to carry an ultimate shear stress of $0.2f'_c$ with zero horizontal load. It was subjected to vertical load only, to determine whether the presence of the horizontal load N_u had influenced the maximum shear stress attained in Corbels F4 and F4A.

The ultimate shear stress developed in Specimen G4 was also $0.14f'_c$, indicating that the unexpected low value of v_u in Specimens F4 and F4A did not result from the action of N_u . Neither the main tension reinforcement nor the stirrup reinforcement developed their yield strengths in Corbels F4, F4A, and G4. These corbels appeared to fail by shear-compression of the concrete in a beam type shear failure.

In push-off tests of all-lightweight concrete previously reported⁶ (i.e., $a/d = 0$), it was possible to develop a maximum shear transfer stress of $0.2f'_c$. It is apparently necessary to define the upper limit of v_u for structural lightweight concrete corbels in terms of a/d . It was decided to check whether a shear stress of $0.14f'_c$ could still be devel-

oped at $a/d = 1.0$ if the main reinforcement yielded.

Specimen J4 ($a/d = 1.0$) was designed to have $v_u = 0.14f'_c$, $N_u/V_u = 1.0$ and a moment capacity (based on the reinforcement yield strength) corresponding to the design ultimate shear. This corbel developed an ultimate shear stress of $0.13f'_c$ and the reinforcement just reached its yield point at failure.

Series H—The sand and gravel concrete corbels of this series were designed to have an ultimate shear strength of 53.4 kips with $N_u/V_u = 0.68$. This corresponds to the upper limit shear stress of $0.3f'_c$ specified when using Eq. (16) for shear design, instead of the shear-friction equation.

Materials and fabrication

The sand and gravel concrete was made from Type III portland cement, sand, and $\frac{3}{4}$ -in. maximum size gravel, in the proportions 1.0:2.77:3.44 by weight.

The all-lightweight concrete was made from the same coated aggregate used in the study⁶ of shear transfer strength. The concrete was made from Type I portland cement, coarse and fine lightweight aggregate in the proportions 1.0:1.48:2.10. In both concretes, water was added sufficient to produce a 3-in. slump. Approximately 6 percent of air was entrained in the concrete. The sand and gravel concrete was moist cured 1 day and then cured in the air of the laboratory until test at age 4 days.

The lightweight concrete was moist cured 7 days and then cured in air until test at 28 days. (This is the standardized curing procedure for tests of structural lightweight concrete.)

The deformed reinforcing bars used for the main tension reinforcement and for the column reinforcement conformed to ASTM Specification A615. The #2 bars used for stirrup reinforcement had deformations similar to the larger bars conforming to ASTM Specification A615.

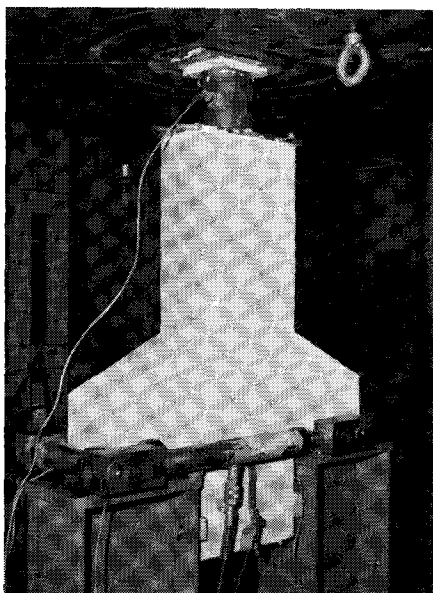


Fig. 4. Arrangement for test of corbel specimens.

A stiff metal jig was used to hold the bearing plates in alignment with one another and the correct distance apart, while they were being welded to the main tension reinforcement. In the form, the bearing plates were screwed to metal supports to ensure their correct alignment and location.

Testing arrangements and instrumentation

For convenience, the specimens were tested in an inverted position, as may be seen in Fig. 4. The specimens of Series A, B, and G were supported directly on the plain bearing plates, through free rollers resting on the tops of the two legs of the U-shaped supporting frame. The vertical load was applied by a Baldwin hydraulic testing machine through an SR4 gage load cell located concentrically on the top of the column.

The specimens of Series C, D, E, F, H, and J were also supported on free rollers; but through an intermediate bearing plate having a projection on its

upper face which mated with the $\frac{1}{2}$ in. deep groove in the bearing plate welded to the corbel reinforcement. Short stub axles projected from each end of the intermediate bearing plates, the centers of the stub axles being at the level of the horizontal face of the corbel.

The horizontal force was applied to the corbels through these stub axles by two 20-kip capacity hydraulic rams, arranged one in front and one behind the specimen, as may be seen in Fig. 4. The horizontal force was monitored by two links instrumented with SR4 gages so as to act as transducers. The vertical load was applied by the testing machine in the same way as for Series A and B.

The average deflection of the corbels at their loading points was measured using a linear differential transformer, attached to the column at its centerline. Resistance strain gages were used to measure the strain in the main tension reinforcement at the interfaces between the column and the corbels in all tests.

In Series F, G, H, and J the strain in the stirrup reinforcement was measured at points where the stirrups crossed a line joining the center of the bearing plate and the intersection of the sloping face of the corbel and the column face. The load cell, transducer links, differential transformer, and strain gages were monitored continuously during the tests by a Sanborn strip chart recorder.

In Series E through J, the maximum width of crack was measured at an arbitrary "service load" of $0.55 V_u$ (design) and $0.5N_u$.

Testing procedure

The specimens of Series A and B were subjected to an incrementally increasing load until failure occurred.

The specimens of Series C and D were first subjected to the horizontal force N_u . This horizontal force was then maintained constant while the vertical load was increased incrementally, until failure occurred. It was considered that

applying the full ultimate horizontal load before applying the vertical load was the most severe loading condition to which a corbel could be subjected, and that the measure of strength so obtained would therefore be conservative.

In the remaining tests, it was decided to try and obtain some measure of serviceability as well as strength, by measuring crack widths at service load. It was arbitrarily decided to consider that the dead load shear V_D and the live load shear V_L were equal. Then, if $\phi = 0.85$, we have $V_D = V_L = 0.275 V_u$.

Also, since ACI 318-71 requires that the horizontal force be treated as a live load, the service horizontal force $N = 0.5N_u$. The following loading sequence was used:

- (1) $N = 0$, V increased incrementally to $V_D = 0.275 V_u$.
- (2) $V = V_D$, N increased incrementally to $0.5 N_u$.
- (3) $N = 0.5N_u$, V increased incrementally to $V_D + V_L = 0.55 V_u$.
- (4) $V = V_D + V_L$, N increased incrementally to N_u .
- (5) $N = N_u$, V increased incrementally until failure occurred.

The maximum crack widths at $V = V_D + V_L$ and $N = 0.5 N_u$ were recorded, this being regarded as the service load.

In all the tests the cracks were marked at each increment of loading.

Test Results

Specimen behavior

Specimens subject to vertical load only—The first cracks to form were flexure cracks, which propagated from the intersection of the column face and the horizontal face of the corbel. These cracks penetrated about halfway through the depth of the corbel at about half the ultimate shear, at which time diagonal tension cracks appeared in the corbels.

These cracks were aligned roughly

along a line running from the intersection of the sloping face of the corbel and the column face, to a point between the inner edge of the bearing plate and the center of the bearing plate. The initial length of these cracks was about one-third the effective depth. As the applied shear was further increased, the diagonal tension cracks increased in length, at first rapidly, then much more slowly as the ultimate load was approached.

In the case of the corbels without stirrups, essentially only one diagonal tension crack formed in each corbel and a very sudden failure occurred when the concrete at the head of these cracks sheared through. This type of diagonal tension failure was referred to by Kriz and Rath² as a "diagonal splitting" failure.

In the case of the corbels with horizontal stirrups, additional inclined cracks occurred, usually starting as flexure cracks at the horizontal face of the corbel. The failure of Corbel B1 was classified as a flexural failure, since it was characterized by wide opening of the flexural cracks, while the diagonal tension cracks remained fine.

The failures of the remaining corbels of Series B were classified as "beam shear" type failures, i.e., a shear failure of the type which occurs in a reinforced concrete beam with stirrup reinforcement. In this case, the flexure cracks remained fine and failure was characterized by widening of one or more diagonal tension cracks and the shear-compression failure of the concrete near the intersection of the sloping corbel face and the column face. Failure was quite abrupt, but less brittle and with more warning than in the case of the diagonal tension failures of the corbels without stirrups. In all cases, the deflections were very small and provided no warning of failure.

Specimens subject to both vertical and horizontal loads—Two types of

crack were caused by horizontal force:

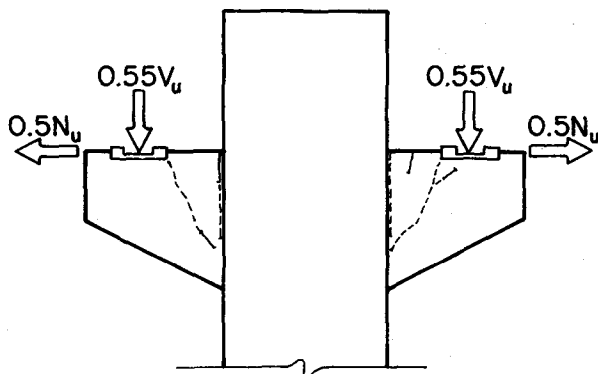
(1) More or less vertical cracks due to the direct tension stresses produced in the concrete by the horizontal force, and

(2) Cracks approximately aligned with the main tension reinforcement and evidently caused by the splitting action of the bar deformations due to the high bond stresses caused by the transfer of some of the horizontal force from the main tension reinforcement to the surrounding concrete.

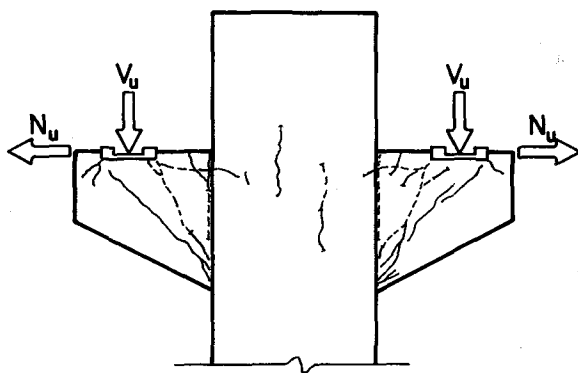
The first cracks to form due to vertical loads were flexure cracks, which either occurred independently or as extensions of the more or less vertical cracks caused by the horizontal force. Diagonal tension cracks initiated at between 40 and 75 percent of the ultimate load, the average diagonal tension cracking shear being 58 percent of ultimate. In some cases, they propagated from the end of a vertical crack caused by the horizontal force, but in other cases, they initiated independently and cut across existing vertical cracks caused by the horizontal forces.

A typical example of the development of cracks is shown in Fig. 5. The dashed lines indicate cracks which occurred when the horizontal load was increased. Full lines are cracks which occurred when the vertical load was increased.

Corbel Specimen D1 failed in flexure with wide opening of the flexure crack adjacent to the column face as the main tension reinforcement yielded, the diagonal tension cracks remaining fine. The failures of all the other specimens subject to both vertical and horizontal forces (except Specimen E4) were classified as "beam shear" type failures and were characterized by widening of the diagonal tension cracks at failure and shear compression failure of the concrete adjacent to the intersection of the sloping face of the corbel and the column face.



Cracking at Service Load



Cracking at Ultimate

Cracking during application of vertical load ———
 Cracking during application of horizontal load - - - -

**Fig. 5. Typical development of cracking
 (Series E through H).**

However, as may be seen from Tables 4 through 8, in most cases the main tension reinforcement yielded before failure occurred. In the case of Specimen E4, a brittle fracture of the reinforcement occurred as the reinforcement yielded. This was caused by embrittlement of the reinforcement due to the welding of the bearing plates onto the reinforcement. Subsequent specimens were annealed after welding to eliminate this problem.

The maximum width of crack at service load in the specimens of Series F, G, H, and J was less than 0.01 in. in all cases. The actual widths are shown in Tables 6 through 8. Complete records of deflection, reinforcement strain and cracking have been reported elsewhere.^{7,8}

Ultimate strength

The loads acting at failure, i.e., the horizontal force N_u and the ultimate

shear force V_u (test) are shown in Tables 4 through 8, together with the nominal ultimate shear stress v_u (test) = V_u (test)/ bd and the shear force V_y (test) which was acting at yield of the main reinforcement. The type of failure exhibited by each specimen is also indicated in these tables as follows:

D.T. = diagonal tension failure

F = flexural failure

B.S. = "beam shear" type failure

Discussion of test results

Minimum stirrup reinforcement—The brittle and complete diagonal tension failure of the Series A corbels without

stirrups, provides additional support for Kriz and Rath's contention that all corbels should be reinforced with horizontal stirrups in addition to the main tension reinforcement, in order to eliminate the possibility of this type of failure occurring.

Except for Specimen B3, the remaining corbels all had horizontal stirrups, the yield strength of which was half the yield strength of that part of the main tension reinforcement not resisting the horizontal force N_w i.e.,

$$A_h f_{yy} = 1/2(A_s f_y - N_y)$$

In all these corbels which were made

Table 4. Test data—Series A & B (all forces in kips).

	Specimen No.	A2 ⁽¹⁾	A3 ⁽¹⁾	B1	B2	B3A
1.	N _u	0	0	0	0	0
2.	V _u (test)	35.6	28.0	47.0	38.9	42.1
3.	v _u (test)(psi)	664	526	870	725	791
4.	v _u /f' _c	0.18	0.14	0.24	0.21	0.19
5.	V _y (test)	*	*	39.7	+	*(2)
6.	V' _u (test)	35.6	28.0	39.7	38.9	42.1
7.	V _u (S.F.)	39.4	41.0	39.2	37.0	42.6
8.	V _u (Mod.S.F.)	44.5	61.5	47.5	54.9	66.6
9.	V(flex)	39.2	44.6	41.1	39.0	44.1
10.	V ₁ (calc)	39.2	41.0	39.2	37.0	42.6
11.	V _u (test) V ₁ (calc)	0.91	0.68	1.20	1.05	0.99
12.	V' _u (test) V ₁ (calc)	0.91	0.68	1.01	1.05	0.99
13.	V ₂ (calc)	39.2	44.6	41.1	39.0	44.1
14.	V _u (test) V ₂ (calc)	0.91	0.63	1.14	1.00	0.95
15.	V' _u (test) V ₂ (calc)	0.91	0.63	0.97	1.00	0.95
16.	Failure Type	D.T.	D.T.	F.	B.S.	B.S.

* Did not yield

+ Strain at failure not known

(1) No stirrup reinforcement

(2) Main reinforcement very close to yield at failure

"D.T." denotes a diagonal tension failure

"F" denotes a flexural failure

"B.S." denotes a beam-shear type failure

Table 5. Test data—Series C & D (all forces in kips).

	Specimen No.	C1	C2	C2A	C3	D1	D2	D3
1.	N_u	32.0	34.0	30.0	30.5	30.0	30.0	30.0
2.	V_u (test)	44.0	40.0	40.5	37.6	28.0	34.0	32.8
3.	v_u (test)(psi)	826	751	760	706	525	638	616
4.	v_u/f'_c	0.21	0.20	0.21	0.16	0.13	0.17	0.17
5.	V_y (test)	38.0	+	(1)	*(2)	19.0	24.2	25.0
6.	V'_u (test)	38.0	40.0	-	37.6	19.0	24.2	25.0
7.	V_u (S.F.)	42.6	39.6	39.5	42.3	29.7	40.5	39.4
8.	V_u (Mod.S.F.)	48.2	58.1	54.7	69.5	38.2	48.7	57.9
9.	V (flex)	36.2	37.1	33.2	40.1	21.8	24.2	26.3
10.	V_1 (calc)	36.2	37.1	33.2	40.1	21.8	24.2	26.3
11.	$\frac{V_u(\text{test})}{V_1(\text{calc})}$	1.21	1.08	1.22	0.94	1.28	1.40	1.25
12.	$\frac{V'_u(\text{test})}{V_1(\text{calc})}$	1.05	1.08	-	0.94	0.87	1.00	0.95
13.	V_2 (calc)	36.2	37.1	33.2	40.1	21.8	24.2	26.3
14.	$\frac{V_u(\text{test})}{V_2(\text{calc})}$	1.21	1.08	1.22	0.94	1.28	1.40	1.25
15.	$\frac{V'_u(\text{test})}{V_2(\text{calc})}$	1.05	1.08	-	0.94	0.87	1.00	0.95
16.	Failure Type	B.S.	B.S.	B.S.	B.S.	F.	B.S.	B.S.

* Did not yield + Strain at failure not known

(1) Yielded, but V_y not known

(2) Main reinforcement very close to yield at failure

"B.S." denotes a beam-shear type failure

"F." denotes a flexural failure

Table 6. Test data—Series E (all forces in kips)

	Specimen No.	E1	E2	E3	E4
1.	N_u	32.5	34.4	35.7	35.1
2.	V_u (test)	55.0	46.0	48.5	35.5
3.	v_u (test)(psi)	1240	1035	1095	800
4.	v_u/f'_c	0.31	0.23	0.26	0.20
5.	V_y (test)	50.5	35.0	37.5	33.2
6.	V'_u (test)	50.5	35.0	37.5	33.2
7.	V_u (S.F.)	36.0	35.5	35.5	35.5
8.	V_u (Mod.S.F.)	42.1	42.8	56.1	53.9
9.	V (flex)	69.9	34.8	36.0	35.1
10.	V_1 (calc)	36.0	34.8	35.5	35.1
11.	$\frac{V_u(\text{test})}{V_1(\text{calc})}$	1.53	1.32	1.37	1.01
12.	$\frac{V'_u(\text{test})}{V_1(\text{calc})}$	1.40	1.01	1.05	0.95
13.	V_2 (calc)	42.1	34.8	36.0	35.1
14.	$\frac{V_u(\text{test})}{V_2(\text{calc})}$	1.31	1.32	1.35	1.01
15.	$\frac{V'_u(\text{test})}{V_2(\text{calc})}$	1.20	1.01	1.04	0.95
16.	Failure Type	B.S.	B.S.	B.S.	(1)
17.	Service Load Crack Width (in)	0.002	0.003	0.003	0.003

(1) Brittle fracture of main tension reinforcement

"B.S." denotes a beam-shear type failure

Table 7. Test data—Series F, G & J (all forces in kips).

	Specimen No.	F2	F3	F4	F4A	G4	J4
1.	N_u	35.6	35.7	35.4	35.5	0	25.0
2.	V_u (test)	36.5	24.0	24.0	23.5	24.0	21.5
3.	v_u (test)(psi)	820	540	540	530	530	485
4.	v_u/f'_c	0.22	0.15	0.13	0.14	0.14	0.13
5.	V_y (test)	28.0	*	*	*	*	21.5
6.	V'_u (test)	28.0	24.0	24.0	23.5	24.0	21.5
7.	V_u (S.F.)	33.0	33.1	35.5	33.0	34.0	22.6 ⁽¹⁾
8.	V_u (Mod.S.F.)	33.0	33.1	35.5	33.0	34.0	22.6 ⁽¹⁾
9.	V (flex)	36.2	35.4	35.4	33.0	34.0	24.8
10.	V_1 (calc)	33.0	33.1	35.4	33.0	34.0	22.6
11.	$\frac{V_u(\text{test})}{V_1(\text{calc})}$	1.11	0.73	0.68	0.71	0.71	0.95
12.	$\frac{V'_u(\text{test})}{V_1(\text{calc})}$	0.85	0.73	0.68	0.71	0.71	0.87
13.	V_2 (calc)	33.0	33.1	35.4	33.0	34.0	22.6
14.	$\frac{V_u(\text{test})}{V_2(\text{calc})}$	1.11	0.73	0.68	0.71	0.71	0.87
15.	$\frac{V'_u(\text{test})}{V_2(\text{calc})}$	0.85	0.73	0.68	0.71	0.71	0.87
16.	Failure Type	B.S.	B.S.	B.S.	B.S.	B.S.	B.S.
17.	Service Load Crack Width(in)	0.004	0.005	0.008	0.008	0.008	0.005

* Did not yield

(1) v_u limited to $0.14f'_c$ in this case

"B.S." denotes a beam-shear type failure

Table 8. Test data—Series H (all forces in kips).

	Specimen No.	H1	H2	H3	H3A	H3B
1.	N_u	35.3	35.7	35.6	36.0	35.6
2.	V_u (test)	67.0	50.0	47.4	39.6	46.1
3.	v_u (test)(psi)	1510	1125	1065	890	1040
4.	v_u/f'_c	0.39	0.29	0.28	0.23	0.27
5.	V_y (test)	44.9	44.5	42.5	39.6	45.7
6.	V'_u (test)	44.9	44.5	42.5	39.6	45.7
7.	V_u (S.F.)	34.8	34.9	34.2	35.2	35.9
8.	V_u (Mod.S.F.)	48.1	52.3	51.3	50.7	50.9
9.	V (flex)	82.0	53.1	52.4	53.5	52.4
10.	V_1 (calc)	34.3	34.9	34.2	35.2	33.9
11.	$\frac{V_u(\text{test})}{V_1(\text{calc})}$	1.93	1.43	1.38	1.13	1.36
12.	$\frac{V'_u(\text{test})}{V_1(\text{calc})}$	1.29	1.28	1.24	1.13	1.35
13.	V_2 (calc)	48.1	52.3	51.3	52.7	50.9
14.	$\frac{V_u(\text{test})}{V_2(\text{calc})}$	1.39	0.96	0.92	0.75	0.91
15.	$\frac{V'_u(\text{test})}{V_2(\text{calc})}$	0.93	0.85	0.83	0.75	0.90
16.	Failure Type	B.S.	B.S.	B.S.	B.S.	B.S.
17.	Service Load Crack Width (in)	0	0.002	0.003	0.004	0.004

"B.S." denotes a beam-shear type failure

of sand and gravel concrete, and for which reinforcement strains were obtained, it is known that the stress in the main tension reinforcement at failure was equal to or very close to the yield point of the steel.

Furthermore, the failure was much less abrupt than in the case of the corbels without stirrups. (The behavior of the all-lightweight concrete corbels will be discussed later.)

It is considered that this behavior indicates that the amount of horizontal stirrup reinforcement provided in these corbels is adequate to eliminate premature diagonal tension failures and to permit the potential strength of the main tension reinforcement to be developed.

Providing the yield point of the stirrup reinforcement is at least equal to that of the main tension reinforcement, the following appears to be an appropriate "minimum stirrup reinforcement" requirement:

"Closed stirrups or ties parallel to the main tension reinforcement, having a total cross-sectional area A_h not less than $0.50 (A_s - N_u/f_u)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to the main tension reinforcement."

This amount of stirrup reinforcement will prevent premature diagonal tension failure of the corbel and will permit the yield strength of the main tension reinforcement to be developed. However, the failure of the corbel may be either a flexural failure or a beam shear type failure after yield of the flexural reinforcement. Either of these modes of failure is considered acceptable, since the full strength of the main tensile reinforcement is developed.

Ultimate strength of sand and gravel concrete corbels—Since Specimens A2 and A3 without stirrups failed in diagonal tension, it is appropriate to compare their strengths with that predicted by

Eq. (14). This equation predicts ultimate shear strengths of 32.2 kips and 32.4 kips, respectively, for Specimens A2 and A3. These compare reasonably well with the test values of 35.6 and 28.0 kips, considering the scatter inherent in the diagonal tension failure strength of reinforced concrete beams.

From the viewpoint of design practice, the ultimate strength results of most interest are those of the specimens which satisfied the minimum stirrup reinforcement requirement proposed above. In Tables 4 through 8, the measured ultimate shear strengths of these corbels have been compared with the ultimate shear strength calculated in various ways:

(a) Using the shear-friction provisions of Section 11.15 of ACI 318-71, but setting the capacity reduction factor ϕ equal to unity. The term $A_v f_{vy}$ was taken as equal to

$$(A_s f_y - N_u) + A_h f_{vy}$$

This calculated shear strength is referred to as V_u (S.F.). (See Line 7 in Tables 4 through 8.) In the case of the all-lightweight concrete corbels, the value of μ was multiplied by the coefficient 0.75 as previously proposed.

(b) Using the modified shear-friction equations previously proposed.^{3,6}

(i) For sand and gravel concrete

$$v_u = 0.8(\rho f_y + \sigma_{Nx}) + 400 \text{ psi (16)}$$

but not greater than $0.3f'_c$.

(ii) For all-lightweight concrete

$$v_u = 0.8(\rho f_y + \sigma_{Nx}) + 200 \text{ psi (17)}$$

but not greater than $0.2f'_c$ nor 800 psi. where $(\rho f_y + \sigma_{Nx})$ was taken as

$$(A_s f_y + A_h f_{vy} - N_u) / bd$$

This calculated shear strength is referred to as V_u (Mod. S.F.) (see Line 8 of Tables 4 through 8).

The ultimate vertical load corresponding to flexural failure, $V(\text{flex})$, is given by

$$V(\text{flex}) = [M_u - N_u(h - d)] / a \quad (18)$$

The moment of resistance of the corbel-column interface plane, M_u , was

based on a flexural reinforcement strength equal to $(A_s f_y - N_u)$, and use of the equivalent rectangular stress distribution, as defined in Section 10.2 of ACI 318-71. Any contribution of the horizontal stirrup reinforcement to flexural strength was neglected (see Line 9 of Tables 4 through 8).

$V_1(\text{calc})$ shown on Line 10 of the tables is the lesser of $V_u(\text{S.F.})$ and $V(\text{flex})$. This method of calculating the shear capacity of a corbel corresponds to the design procedure proposed earlier in this paper.

$V_2(\text{calc})$ shown on Line 13 of the tables is the lesser of $V_u(\text{Mod. S.F.})$ and $V(\text{flex})$. This method of calculating the shear capacity of a corbel corresponds to the design procedure proposed earlier, modified by using the modified shear-friction relationship in place of the shear-friction provisions of ACI 318-71.

It can be seen from Line 11 of the tables that $V_1(\text{calc})$ is a generally conservative estimate of the ultimate shear strength, the average value of $V_u(\text{test})/V_1(\text{calc})$ for Series B, C, D, and E being 1.20, and for Series H being 1.45.

On Line 5 of the tables is given V_y , the vertical load at which the main tension reinforcement yielded. It can be seen that in a number of cases, yield of the reinforcement occurred at a vertical load considerably less than the ultimate load. This behavior tends to be greatest for low values of a/d and for smaller reinforcement ratios, such as Series D. This behavior was also pointed out by Kriz and Rath².

The load factors specified in ACI 318-71 are intended to provide a safety margin against excessive cracking due to yield of the reinforcement, as well as to provide safety against collapse. The flexural ultimate strength of a reinforced concrete beam calculated according to Section 10.2 of ACI 318-71, cor-

responds very closely to the moment at yield of the flexural tension reinforcement in a reinforced concrete beam of usual proportions.

To provide the same safety margin against wide cracking in corbels due to yield of the main tension reinforcement when using the load factors of ACI 318-71, as is provided in the case of ordinary reinforced concrete beams using the same load factors, it is proposed that the "useful ultimate strength," V'_u be taken as the vertical load at yield of the main tension reinforcement or the ultimate load when the main tension reinforcement does not yield.

The measured useful ultimate strengths V'_u (test) of the corbels tested is given on Line 6 of Tables 4 through 8. It can be seen from Line 12 of the tables that $V_1(\text{calc})$ is a good predictor of the useful ultimate strength V'_u , the average value of $V'_u(\text{test})/V_1(\text{calc})$ for Series B, C, D, and E being 1.03 and for Series H being 1.26.

These results indicate that the proposed method of corbel design will ensure both adequate strength and adequate serviceability.

The values of $V_u(\text{test})/V_2(\text{calc})$ shown on Line 14 of the tables indicate that $V_2(\text{calc})$ is also a good predictor of the ultimate strength $V_u(\text{test})$, although slightly less conservative than $V_1(\text{calc})$. The average value of $V_u(\text{test})/V_2(\text{calc})$ is 1.18 for Series B, C, D, and E and 0.99 for Series H. The result obtained for Series H is the more significant since this series of corbels was designed for maximum shear stress allowed by the modified shear-friction equation, i.e., $0.3f'_c$. With the exception of Corbel H3A, this shear stress was exceeded or closely approached.

(The reason for the low value of ultimate strength given by Corbel H3A is not known. As far as could be determined, its physical properties were almost identical with those of Specimens

H3 and H3B, which yielded higher ultimate strengths. The yield strength V_y was also low for this specimen.)

On Line 15 of the tables are given values of $V'_u(\text{test})/V_2(\text{calc})$. The average value of $V'_u(\text{test})/V_2(\text{calc})$ is 1.00 for Series B, C, D, and E and 0.85 for Series H. The average for Series H is low because the values of $V_y(\text{test})$ are all low relative to $V(\text{flex})$.

This probably indicates that at high ultimate shear stresses such as $0.3f'_c$, the interaction between shear and direct stress in the flexural compression zone reduces the average normal stress at failure to a value significantly less than that corresponding to the parameters of the equivalent rectangular stress distribution specified in Section 10.2.7 of ACI 318-71.

This would lead to a reduction in the internal lever arm of the interface plane between the corbel and the column, with a consequent reduction in the moment at yield of the reinforcement.

The safety factor against yield of the reinforcement will therefore be less if the modified shear-friction equation is used in design, and the ultimate shear stress is made equal to the maximum allowed, i.e., $0.3f'_c$.

However, it can be seen from Line 17 of Tables 6 and 8 that the average value of the maximum crack width at service load was the same for Series H and E, even though the shear stress at service load was 50 percent higher in Series H than in Series E. In both cases, the average maximum crack width at service load was very small, being approximately 0.003 in.

Because the crack widths in the Series H corbels were so small at service load and the average value of $V_u(\text{test})/V_2(\text{calc})$ for Series H was equal to 0.99, it is considered that the modified shear-friction equation, Eq. (16), could be used for shear design in the corbel design procedure proposed,

in place of the shear-friction provisions of ACI 318-71.

Ultimate strength of lightweight concrete corbels—Data obtained in the tests of the all-lightweight concrete corbels of Series F, G, and J are shown in Table 7. As already mentioned in the discussion of the design of the specimens of Series F, it was found that the maximum shear stress attainable decreased as a/d increased, the failure at larger values of a/d occurring by shear compression of the concrete before the yield strength of both the main tension reinforcement and the stirrup reinforcement could be developed.

The variation of the useful ultimate shear stress with the shear span to depth ratio a/d is shown in Fig. 6. The value shown at a/d equal to zero is the value of $0.2f'_c$ previously developed in push-off tests of all-lightweight concrete.⁶

It is proposed that in the shear design of all-lightweight concrete corbels, (using either the shear-friction equation or the modified shear friction equation), the maximum ultimate shear stress v_u (max.) be limited to

$$v_u(\text{max.}) = \left(0.2 - 0.07 \frac{a}{d} \right) f'_c \quad (19)$$

but not more than $\left(800 - 280 \frac{a}{d} \right)$ psi.

It can be seen in Fig. 6 that use of this equation will result in a close estimate of the maximum useful shear stress obtainable in an all-lightweight concrete corbel. This limiting value would replace the limiting value of " $0.2f'_c$ nor 800 psi" previously proposed for all-lightweight concrete when using either the shear-friction equation or the modified shear-friction equation, Eq. (17).

In Table 9, a revised comparison is made of the measured and calculated strengths of the all-lightweight concrete

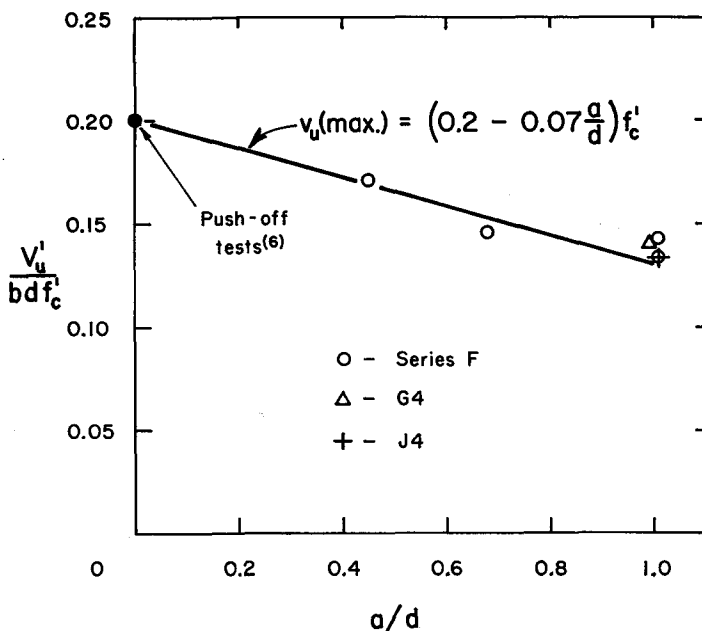


Fig. 6. Variation of maximum obtainable useful ultimate shear stress in all-lightweight concrete corbels, with the shear span to depth ratio, a/d .

Table 9. Revised comparison of measured and calculated ultimate strengths of lightweight concrete corbels (all forces in kips).

Specimen No.	F2	F3	F4	F4A	G4	J4
$V_u(\text{test})$	36.5	24.0	24.0	23.5	24.0	21.5
$V_y(\text{test})$	28.0	*	*	*	*	21.5
$V_u'(\text{test})$	28.0	24.0	24.0	23.5	24.0	21.5
$V_u(\text{S.F.})$	27.9	25.3	23.2	21.3	22.2	20.9
$V_u(\text{Mod.S.F.})$	27.9	25.3	23.2	21.3	22.2	20.9
$V(\text{flex})$	36.2	35.4	35.4	33.0	34.0	24.8
$V(\text{calc})^{(1)}$	27.9	25.3	23.2	21.3	22.2	20.9
$\frac{V_u(\text{test})}{V(\text{calc})}$	1.31	0.95	1.03	1.10	1.08	1.03
$\frac{V_u'(\text{test})}{V(\text{calc})}$	1.00	0.95	1.03	1.10	1.08	1.03

* Did not yield

(1) For all these specimens, $V_1(\text{calc}) = V_2(\text{calc}) = V(\text{calc})$. This is because the calculated shear strengths $V_u(\text{S.F.})$ and $V_u(\text{Mod.S.F.})$ are both always governed by the maximum allowable shear stress as given by Eq.(19).

corbels, using Eq. (19) as the upper limit to the ultimate shear stress calculated using both the shear-friction and modified shear-friction equations. It can be seen that use of Eq. (19) leads to a close estimate of the useful ultimate strength of all the all-lightweight concrete corbels tested.

The shear strength of an all-lightweight concrete corbel is less than that of a sand and gravel concrete corbel with identical dimensions, reinforcement, and concrete compressive strength, as may be seen from Series E and F. The reason for this difference in behavior is probably the difference in smoothness of the faces of the diagonal tension cracks which form in the corbels.

The crack faces in the sand and gravel concrete are very rough. This is because the bond strength between the cement paste and the aggregate particles is less than the tensile strength of the aggregate particles, so a crack generally follows the interface of the aggregate particles and the cement paste.

The crack faces in the lightweight concrete were smoother than in the sand and gravel concrete. This is because the bond strength between the cement paste and the lightweight aggregate particles is greater than the tensile strength of the lightweight aggregate, so a crack due to tension passes through the lightweight aggregate particles, and the crack faces are relatively smooth.

Because of the smoothness of the crack faces, it is probable that little or none of the inclined compression force could be transferred across the principal diagonal tension crack in the all-lightweight concrete corbels. Consequently, all of the diagonal compression force would have to pass through the compression zone above the tip of the diagonal tension crack in the all-lightweight concrete corbels.

However, in the sand and gravel

concrete corbels, some of the diagonal compression force was apparently transferred across the principal diagonal tension crack by aggregate interlock effects. (Evidence of this was local compression spalling adjacent to the crack faces at ultimate.)

Consequently, only part of the diagonal compression force would have to pass through the compression zone above the tip of the principal diagonal tension crack. For a given applied shear, the stresses in the compression zone above the tip of the principal diagonal tension crack would therefore be greater in the lightweight concrete corbel than in the sand and gravel concrete corbel.

Conversely, stresses in the compression zone high enough to initiate failure would occur at a lower applied shear in the lightweight concrete corbels than in the sand and gravel corbels.

No tests were made of sanded lightweight concrete corbels, due to limitations of time and resources. It is probable, however, that the maximum ultimate shear stress developable for sanded-lightweight concrete would be similar to that for all-lightweight concrete, because of similarity in shear transfer behavior previously demonstrated.⁶

The following equation has previously⁶ been proposed for the shear transfer strength of sanded lightweight concrete:

$$v_u = 0.8(pf_y + \sigma_{N\pi}) + 250 \text{ psi} \quad (20)$$

but not more than $0.2f'_c$ nor 1000 psi.

By analogy with the case of all-lightweight concrete, it is proposed that when Eq. (20) is being used in the design of corbels, the upper limit to v_u should be changed to

$$\begin{aligned} &\text{"but not more than } \left(0.2 - 0.0 \frac{a}{d}\right) f'_c \\ &\text{nor } \left(1000 - 350 \frac{a}{d}\right)'' \end{aligned}$$

Conclusions for Design

On the basis of the study reported here, the following conclusions are drawn:

1. The design of corbels using the load factors in Section 9.3 of ACI 318-71 should be based on the "useful ultimate strength," in order that an adequate safety margin against wide cracking shall be maintained.

(In evaluating test data, the "useful ultimate strength" is defined as the vertical load at yield of the main tension reinforcement, or the vertical load at failure if yield of the main tension reinforcement does not occur.)

2. Subject to the provision of minimum horizontal stirrup reinforcement according to Conclusion 3 below, the "useful ultimate strength" of corbels subjected to a combination of vertical and horizontal loads can be calculated with satisfactory accuracy by taking it to be the lesser of

(a) The shear strength of the corbel-column interface, calculated using either the shear-friction provisions of Section 11.15 of ACI 318-71* or the modified shear-friction equation;^{*} and

(b) The vertical load corresponding to the development of the flexural ultimate strength of the corbel-column interface, taking into account the additional moment $N_u(h-d)$, imposed on the corbel by the horizontal tension force N_u , and using the provisions of Section 10.2 of ACI 318-71.

3. A minimum amount of horizontal stirrup reinforcement must be provided in corbels to eliminate the possibility of a premature diagonal tension failure. The yield strength of this stirrup reinforcement, $A_k f_y$, should be not less than one-half the yield strength of that part of the main tension reinforcement necessary to resist moment, or one-third the yield strength of the reinforcement

necessary to resist shear, whichever is the greater.

4. The use of the shear-friction provisions of Section 11.15 of ACI 318-71 in the design of corbels, can be extended beyond the limits currently set in Section 11.14 of ACI 318-71. An alternate Section 11.14 is proposed in Reference 9.

5. In the design of all-lightweight concrete corbels for shear, using either the shear-friction or modified shear-friction equation, the ultimate shear stress v_u should not exceed:

$$\left(0.2 - 0.07 \frac{a}{d}\right) f_c$$
$$\text{nor } \left(800 - 280 \frac{a}{d}\right) \text{ psi}$$

6. In the design of sanded-lightweight concrete corbels, using either shear-friction or modified shear-friction, the ultimate shear stress v_u should not exceed:

$$\left(0.2 - 0.07 \frac{a}{d}\right) f_c$$
$$\text{nor } \left(1000 - 350 \frac{a}{d}\right) \text{ psi}$$

Acknowledgments

This study was carried out in the Structural Research Laboratory of the University of Washington. It was jointly supported by the National Science Foundation, through Grant No. GK-33842X, and by the Prestressed Concrete Institute, through its PCI Graduate Fellowship program. Lightweight aggregate was supplied by a member of the Expanded Shale, Clay and Slate Institute.

*Modified for lightweight concrete as proposed here and in Reference 6.

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9. Mattock, A. H., "Design Proposals for Reinforced Concrete Corbels." (To be published in May-June, 1976, *PCI JOURNAL*.)

Discussion of this paper is invited.
Please forward your discussion to
PCI Headquarters by August 1, 1976.

Coming in the Next Issue

The May-June, 1976, *PCI JOURNAL* will contain a follow-up paper by Dr. Alan H. Mattock on "Design Proposals for Reinforced Concrete Corbels."

This paper will present a model code clause and design procedures for corbels. Included also will be design examples for both normal weight and lightweight concrete corbels, using both the shear-friction and modified shear-friction approaches to shear transfer design of the corbel-column interface.

Appendix—Notation

- A_{er} = area of shear plane, sq in.
 A_f = area of reinforcement necessary for flexure, sq in.
 A_h = total area of stirrup reinforcement parallel to main tension reinforcement, sq in.
 A_s = area of main tension reinforcement, sq in.
 A_t = area of reinforcement necessary to resist horizontal tension force N_u , sq in.
 A_{vf} = area of shear-friction reinforcement, sq in.
 a = shear span; distance between a concentrated load and face of support, in.
 b = width of compression face of member, in.
 C = resultant concrete compression force in flexural compression zone
 d = distance from extreme compression fiber to centroid of tension reinforcement, in.
 f'_c = compressive strength of concrete measured on 6×12 -in. cylinders, psi
 f_{vy} = yield point stress of stirrup reinforcement, psi
 f_y = yield point stress of reinforcement, psi
 h = overall depth of corbel at column face
 $j'd$ = distance from centroid of main tension reinforcement to center of action of resultant concrete compression force C , in.
 j_d = distance from centroid of horizontal stirrup reinforcement to center of action of resultant concrete compression force C , in.
 $j'd$ = distance from centroid of flexural tension reinforcement to center of action of resultant concrete compression force if only flexural reinforcement of strength $(A_s f_y - N_u)$ were acting in section, in.
 M_u = ultimate moment of resistance of corbel-column interface plane, in.-kips
 N_u = design ultimate tensile force on corbel acting simultaneously with V_u , kips
 $V(\text{flex}) = [M_u - N_u(h-d)]/a$
 V_u = ultimate shear strength, kips
 V'_u = useful ultimate shear strength, kips
 $= V_y$, or V_u if main tension reinforcement does not yield
 V_y = shear acting at yield of main tension reinforcement, kips
 v_u = nominal ultimate shear stress, psi
 $= V_u/\phi b d$
 μ = coefficient of friction used in shear-friction calculations
 ρ = A_{vf}/A_{er} in modified shear-friction equation
 ρ = A_s/bd in corbels
 ρ_h = A_h/bd
 $\sigma_{N\sigma}$ = externally applied normal stress acting across shear plane, psi (positive if compression, negative if tension)
 ϕ = capacity reduction factor, as defined in Section 9.2 of ACI 318-71

Technical Note

Strand savings using strain compatibility in strength design



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The purpose of this article is to show that strand savings (in terms of additional resisting moment) are possible when one uses the more accurate strain compatibility method allowed by the ACI Code.

A design example compares the results using Eq. (18-3) of the ACI Code and a strain compatibility method. This latter method utilizes charts in the *PCI Design Handbook*.

Both current design practice and the requirements of ACI 318-71¹ require the checking of ultimate flexural strength (strength design) of bonded prestressed concrete members.

The determination of internal resisting moment developed at ultimate capacity is based on the following assumptions:

1. Plane sections remain plane.
2. The stress-strain curve for concrete in compression is replaced by an equivalent rectangular stress block.
3. The stress-strain curve for prestressing steel is nonlinear.

ACI Code Method [Eq. (18-3)]

Satisfying the above three assumptions will result in what is commonly called "strain compatibility." ACI 318-71, however, allows a strength design analysis based on Eq. (18-3) which offers a conservative approximation for Assumption 3.

This approximation, expressed in Eq. (18-3), is as follows:

$$\frac{f_{ps}}{f_{pu}} = 1 - 0.5 \rho_p \frac{f_{pu}}{f'_c}$$

where

f_{ps} = calculated stress in prestressing steel at design load

f_{pu} = ultimate strength of prestressing steel

ρ_p = ratio of prestressed reinforcement

f'_c = specified compressive strength of concrete

Strain Compatibility Method (Using PCI Design Handbook)

The *PCI Design Handbook*² has a useful design chart on p. 5-25, reproduced as Fig. 1, in which f_{ps}/f_{pu} ratios can be evaluated quickly and accurately.

In using the chart, the following is assumed:

1. Normal weight concrete.
2. Effective stress in prestressing steel after loss is approximately 150 ksi.
3. Prestressing steel has an ultimate strength of 270 ksi.

Comparison of Methods

Four typical cases were analyzed: two inverted T-beams and two double-tees.

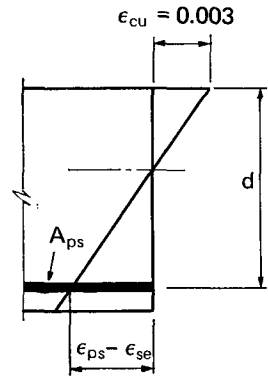
The computational results are summarized in Table 1 which also lists

Table 1. Comparison of strength design steel capacity, strain compatibility vs. approximate Eq. (18-3).

Type of section	b, in.	d, in.	f' _c , ksi	No of strand	ω_p	Ratio f_{ps}/f_{pu}		Percent additional resisting force in strands by strain compatibility
						By strain compatibility	By approximate Eq. (18-3)	
Untopped, Inverted T-Beam	14	31	5	16	0.304	0.903	0.847	6.6
Topped, Inverted T-Beam	70	34.5	4	22	0.094	0.972	0.953	2.0
8 DT 18, Untopped	96	14	5	10	0.062	0.980	0.969	1.1
8 DT 14, Topped	96	13.5	3	8	0.085	0.972	0.957	1.5

$$C \bar{\omega}_p = C \frac{A_{ps}}{bd} \frac{f_{pu}}{f'_c}$$

Values of C	
f'_c	C
3000	0.94
4000	1.00
5000	1.06
6000	1.13
7000	1.21
8000	1.31



STRAND

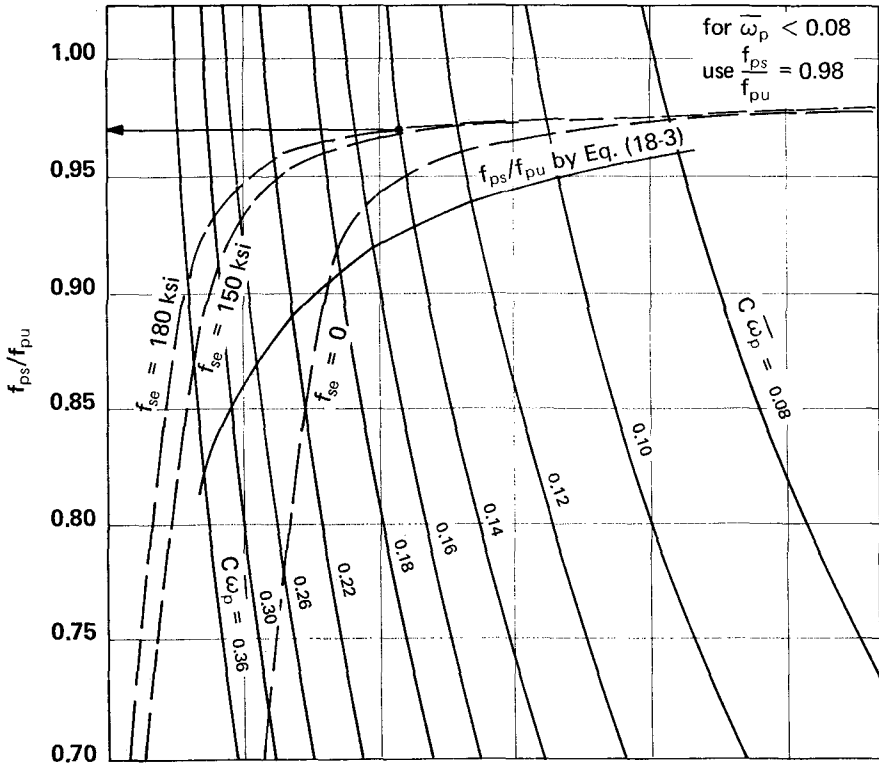


Fig. 1. Values of f_{ps} by stress-strain relation—bonded strand (taken from Fig. 5-25, *PCI Design Handbook*).

the concrete strengths of the compression stress block and other section properties.

A sample calculation is given comparing the results from the *PCI Design Handbook* chart and the approximate ACI Code equation.

Conclusions

The savings in strand are most pronounced in the case of untopped, inverted T-beams, which in this case reached 6 percent.

For topped, inverted T-beams or double-tee sections the savings are generally below 2 percent.

Design Example

A comparison is now made of the results obtained using the *PCI Design Handbook* chart and the approximate ACI Code equation. An untopped, inverted T-beam is used as a design example in the computations. The section used is shown in Fig. 2.

$b = 14$ in., $d = 31$ in.

$f'_c = 5$ ksi; $f_{ps} = 270$ ksi.

No. of strand: sixteen $\frac{1}{2}$ -in. diameter.

Total strand area:

$$A_{ps} = 16 \times 0.1531 = 2.4496 \text{ sq in.}$$

Method 1—From the *PCI Design Handbook* chart (Fig. 1): $C = 1.06$.

$$\bar{\omega}_p = [2.4496 / (14 \times 31)] (270/5) = 0.304$$

$$C\bar{\omega}_p = 1.06 \times 0.304 = 0.322$$

Entering the chart with "0.322," one gets $f_{ps}/f_{pu} = 0.903$.

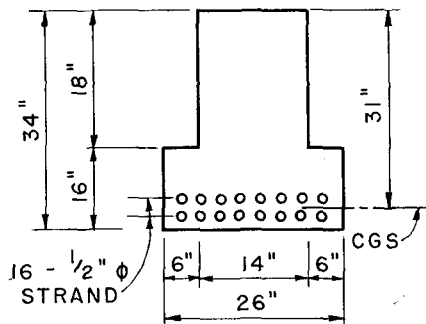


Fig. 2. Cross section of inverted T-beam.

Method 2—Using the approximate ACI Eq. (18-3):

$$f_{ps}/f_{pu} = 1 - 0.5 [2.4496 / (14 \times 31)] (270/5) = 0.847$$

Comparing the results from Methods 1 and 2, it is readily seen that the strain compatibility approach will result in an additional resisting force in the strands of

$$(0.903/0.847) - 1 = 0.066 \text{ (or 6.6\%)}$$

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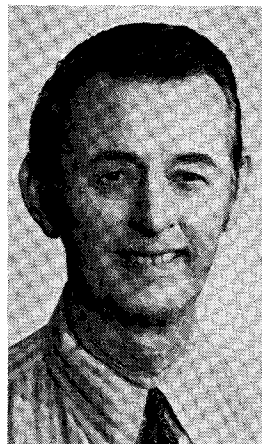
Discussion of this paper is invited. Please forward your comments to PCI Headquarters by August 1, 1976.

Splicing technique solves major precast pile salvaging operation

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The author, who was a consultant on this job, describes a splicing method which salvaged a sure loss while furnishing piles of adequate capacity for the foundation of a multilevel garage in Bangkok, Thailand.



A unique pile splicing technique saved both time and money on a major precast concrete pile salvaging operation in Bangkok, Thailand. It is estimated that the salvaging operation gained at least 30 working days on the construction schedule and saved about \$16,000.

The specific job confronting the consultant was to build a secure foundation for a six-story parking garage for the Phase II Oriental Hotel in Bangkok.

In view of the anticipated high dead and live loads for the garage and the existing poor foundation soils, it was estimated that 80 precast concrete piles would be needed underneath the superstructure.

To provide this number of brand

new piles would have been a prohibitively expensive undertaking. Furthermore, it was not physically possible to transport full-length piles through the narrow streets of Bangkok to the building site. The other alternative was to ship the piles in by barge or pontoon via the Chao Pyia river. However, because construction was well advanced, there was no access to the river bank.

Nevertheless, prior to this job, the consultant was aware that several thousand precast concrete piles were driven in connection with the Bangkok harbor extension and berthing docks.

Many of these 25-m (82 ft) long piles were broken at their pickup point or approximately at their 17 to 18-m (about 60 ft) point. These damaged

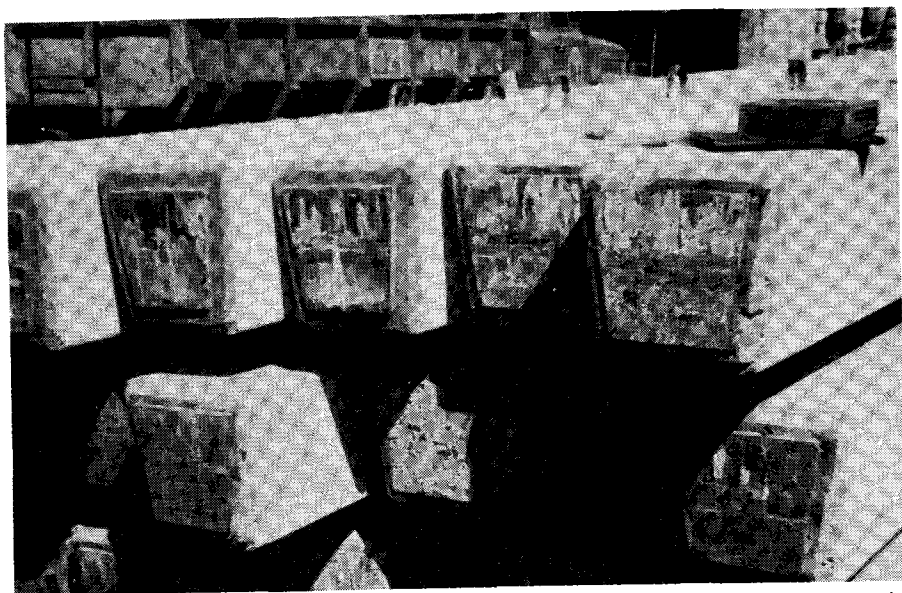


Fig. 1. Stockpile of reclaimed piles (tip end only). The final weld plate, approximately 0.3 m (1 ft) square, kept the pile top in perfect condition while using the heavy hammer. The anchor rods holding the plates were epoxied into predrilled holes. The pile end was mortared at a bevel mainly for appearance and to leave only the weld surface exposed. The top unit was exactly the same minus the steel point for perfect fit.

and rejected piles represented a considerable loss of money and waste.

Therefore, a scheme was developed whereby the damaged piles could be salvaged and reused in the garage foundation job.

The major problem was to devise a pile splice which would meet the design requirements of the Oriental Hotel extension.

The existing damaged piles were of an unusually odd shape to meet harbor design requirements. Their cross section was 0.30 x 0.39 x 0.30 m (12 x 15½ x 12 in.), obtained using steel forms.

The first layer of bearing sands at the Oriental Hotel extension was situated at a depth of about 14 m (46 ft). The second layer of bearing sands was at an approximate 26-m (85 ft) depth and was about 3 m (10 ft) thick.

The latter tip elevation was selected

for a trial run. A few test piles were driven and their bearing capacity proved to be excellent. The piles were able to withstand a load in excess of 200 metric tons (440 U.S. tons) versus a design capacity of 70 metric tons (154 U.S. tons).

Solution

The pile splicing technique adopted was a variation of the cement-dowel splice and the Bolognesi-Moretto splice.*

The specific splicing procedure (see also Figs. 1-5) involved the following steps:

1. Broken piles were cut off at their 12.5 m (41 ft) length. Note that some

*For a full description of these splices refer to the report by Robert Bruce, Jr., and David Hebert on "Splicing of Precast Prestressed Concrete Piles—Review and Performance of Piles," published in the September-October 1974 PCI JOURNAL, pp. 70-97.

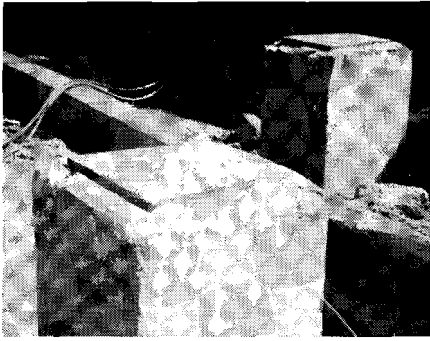


Fig. 2. Lower pile units driven ready for positioning of upper unit and welding. The units were driven to about 0.6 m (2 ft) above grade for easy welding.



Fig. 3. Drop hammer poised above lower pile unit following driving showed no spalling or damage during this stage.



Fig. 4. Checking exact line-up of two pile units prior to welding. Positioning was important to provide uniform weld area. Repair and plate alignment had to be carefully positioned to assure perfect mating. The join-up was relatively simple if preliminary care is exercised. The entire 80 double unit piles were driven without incident.



Fig. 5. This is the four-sided weld being applied while the driving rig holds the upper unit in position and steady. A three-pass weld was used employing mild steel $\frac{1}{4}$ -in. coated rod. The epoxy coat was welded. Welding time took about 30 minutes. During the driving operation and afterwards, no breakage of piles was observed.

of the piles were broken on their tip end *while others were broken on their driving end.*

2. Weld plates 12 mm ($\frac{1}{2}$ in.) thick with four 25-mm (1 in.) diameter steel dowels were drilled into the ends. The weld plate was partially grouted leaving more than 25 mm (1 in.) exposed.

3. The tip pile unit was driven and then the upper pile unit positioned on top. The plates were then ring welded on four sides using a 19-mm ($\frac{3}{4}$ in.) weld. Pile driving was resumed almost immediately. The time involved in positioning the pile units and welding took about 30 minutes. The total driving time comprised $1\frac{1}{2}$ to 2 hours.

The following pile driving criteria were adopted:

1. The weight of the drop hammer was specified at 6.8 metric tons (15 U.S. tons). No adjacent vibration was allowed.

2. The maximum drop was set at 0.4 m (1 ft).

3. The final blow count was to be 40 blows for the last 0.1 m (4 in.).

In general, the blow count while driving ranged from 15 to 20 blows. The first sand layer was reached at 14 m (46 ft) and the blow count rose to 22-25 blows.

Concluding Remarks

By using this splicing technique, we were able to salvage a sizeable loss of damaged piles and still provide a new pile foundation of adequate capacity.

During the pile driving operation no undue problems or breakage were experienced.

The author believes that this splicing technique, with perhaps some adaptation, can be used on other pile foundation jobs.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by August 1, 1976.

Deflection of prestressed concrete flat plates

Edward G. Nawy

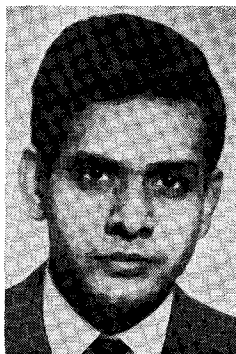
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Considerable advances have been made in the last two decades in the design and construction of prestressed two-way concrete floor systems.

The logical use of flat plates, on both economical and functional grounds, have made prestressing of two-way slabs attractive, particularly as an extension of the experience gained in pre-

stressed lift slab construction in the 1950's and post-tensioned flat plate construction during subsequent years.

In the ensuing years it has become important, both from a serviceability and performance viewpoint, that closer attention be given to the deflection behavior of two-way prestressed concrete continuous plates.

The ACI 318-71 Code¹ and ACI

318-71 Commentary² give only very general guidelines on deflection. In effect, the Code specifies that a span to thickness ratio of 48 to 52 can be permitted provided that "calculations verify that short-term and long-term deflection, camber, and vibration frequency and amplitude are not objectionable."

The work reported herein comprises a theoretical and experimental investigation to study the deflection behavior of four-panel continuous post-tensioned concrete plate models loaded with distributed load.

The panels are 30 x 30 in. square and have a 1 $\frac{1}{16}$ -in. nominal thickness.

The strand was draped parabolically and was distributed either uniformly or banded. In one of the test slabs mild steel reinforcement was added to improve crack control.

Experimental Investigation

Materials

Concrete—Quality concrete was supplied ready-mixed with the adjusted design mix water added at the time of placement. After several trial batches, the prescribed compressive strength of the concrete was set as 5500 psi at 7 days.

High early strength Type III cement was used.

The coarse aggregate comprised natural $\frac{3}{8}$ in. maximum size graded stone and natural sand. Slump was maintained at 3 to 4 in.

Ten 6 x 12-in. standard cylinders were tested for both compressive and tensile splitting strength for each slab. In addition, three 6 x 6 x 22-in. plain concrete beams were cast with each test slab and tested for modulus of rupture.

The design mix per cubic yard of concrete developed in this investigation comprised the following proportions for a 3-in. slump.

893 lbs of high early strength cement (9 $\frac{1}{2}$ bags).

1410 lbs of crushed $\frac{3}{8}$ in. maximum size stone.

1250 lbs of graded sand.

Synopsis

Reports the results of a theoretical and experimental investigation into the deflection behavior of four 60 x 60-in. four-panel continuous prestressed flat plate models.

Each panel of the slab specimens is 30 x 30 in. square and is 1—5/16 in. thick

The plates were subjected to a uniformly distributed load with post-tensioning applied in two directions.

The strand was draped parabolically and was distributed both uniformly and banded. To control cracking, mild steel reinforcement was added in one of the slab specimens.

A rational method, based on an equivalent beam-frame analogy, was developed to calculate deflection and camber in two-way prestressed flat plates.

The Appendix section includes a sample calculation for determining the deflection and camber in one of the test slabs. The experimental results show good correlation with the theory through the elastic range.

The subject of this paper is confined to plate behavior up to the first cracking load. A future paper will deal with the post-cracking behavior of plates.

Table 1. Summary of concrete properties of slab specimens.

Slab Designation	Control Tests Age days	Cylinder Compressive Strength, psi f'_c (f'_{ci})	Tensile Splitting Strength f'_t psi	Modulus of Rupture f_r psi	Slump in.	Temperature at Casting F°	Water Content in Mix gal/yd ³	Material Modulus E_c psi $\times 10^{-6}$
1	2	3	4	5	6	7	8	9
RP-1	9½ (6½)	5730 (4000)	435	545	3½	65	35	4.313 (3.600)
RP-2	9½ (6½)	5570 (4120)	400	513	3½	65	32	4.256 (3.660)
RP-3	9½ (6½)	5600 (4100)	436	526	4	65	31½	4.265 (3.650)
RP-4	9½ (6½)	5900 (4290)	412	562	4	80	29	4.378 (3.730)

32 gallons of water.

No additive or air-entraining agents were used. Table 1 shows the actual water added and the resulting slump.

Post-tensioning steel—Based on the geometric proportions of the slab specimens and scale requirements, a single 0.120-in. diameter prestressing wire strand was used.

The wire specified was ASTM A-421 grade 270 non-stress relieved type. As recommended by the manufacturer's specifications, stress relieving was accomplished by subjecting the wire to a constant 500 C temperature for ½ hr, and then air-cooling.

Tensile tests on the wire control specimens gave an ultimate strength of 283 ksi as shown in the stress-strain plot of Fig. 1. The steel had a yield strength of 272 ksi at 0.2 percent offset and its proportional limit was 224 ksi.

Mild reinforcing steel—High strength steel bars (A-82) at the tension column regions of the slabs were used to reinforce them against shear failure. Also, one test slab (Specimen No. RP-4) was additionally reinforced with some mild steel in order to study crack control. Tension tests on control specimens were made to establish the steel's strength as given in Fig. 2. Its yield strength was 75 ksi.

Plastic tubing—The individual prestressing wires were inserted through 9/64-in. wide diameter plastic tubes. This method eliminated bonding of the wire to the surrounding concrete and facilitated a smooth post-tensioning of the tendons.

Test specimens

Four plate specimens (RP-1 through RP-4), each 5 ft 3 in. x 5 ft 3 in. x 1-5/16 in. in size, were cast and subsequently post-tensioned.

The slab area was subdivided into four panels and the prestressing wires were parabolically draped and made continuous over the central column support (Fig. 3).

Two important variables were included in the plate tests: (1) the arrangement of the prestressing steel; and (2) the addition of mild steel reinforcement in Test Specimen RP-4. This comparison is shown in Fig. 4.

Test Specimen RP-1 had the tendons uniformly distributed in both directions (Fig. 4a).

The second test slab (Specimen RP-2) had its prestressing reinforcement arranged in accordance with ACI 318-71 recommendations. About 67 percent of the strands in both orthogonal directions were placed in the column strip (Fig. 4b).

The third test slab (Specimen RP-3) had

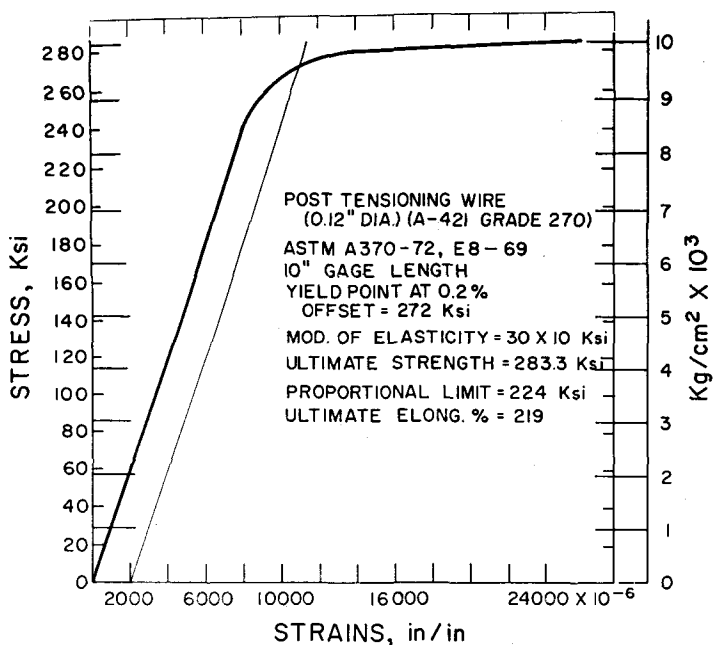


Fig. 1. Stress-strain diagram for prestressing wires.

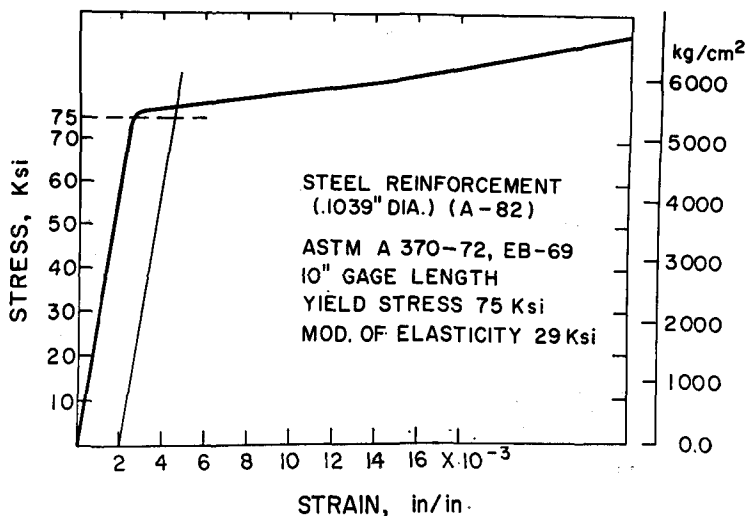


Fig. 2. Stress-strain diagram for mild reinforcing steel.

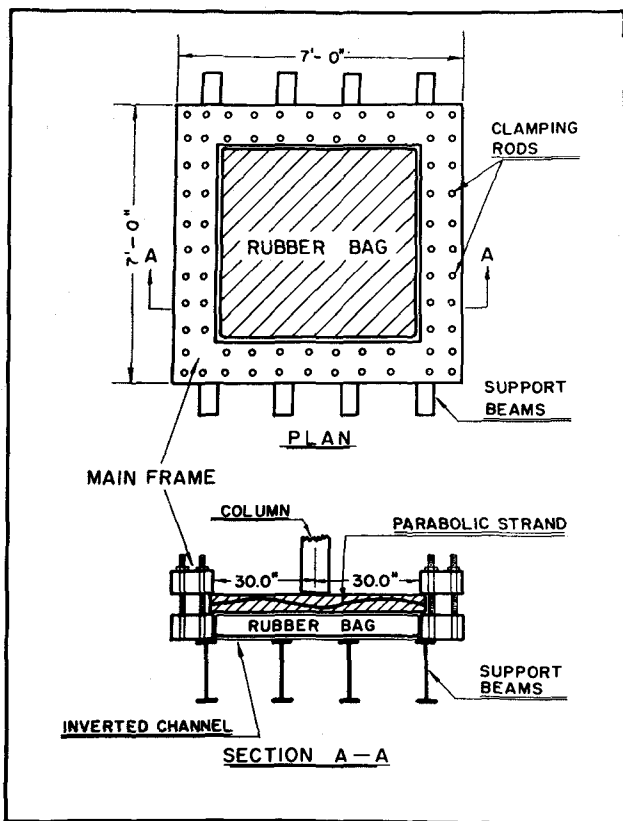


Fig. 3. Plan and section of test setup.

the tendons uniformly distributed in one direction. In the orthogonal direction, almost all the prestressing steel was banded or concentrated well within the column strip (Fig. 4c). This distribution scheme is advantageous because the steel can be easily installed and the system is often economical in practice.

The fourth test slab (Specimen RP-4) had the same wire arrangement as the first test slab (Specimen RP-1). However, smooth high strength reinforcement (0.103 in. diameter) was added evenly distributed at $5\frac{1}{4}$ in. center to center. Table 2 gives the various properties of all the test specimens.

Specimen preparation and test setup

Specimen preparation—Accurately constructed rigid wooden forms of $\pm\frac{1}{16}$ -in. tolerance were used in this investigation. They were appropriately oiled before reinforcement or concrete placement.

The prestressing wires were greased and inserted inside the plastic tubes. The length of the plastic tubes were such that they fitted precisely inside the form edges while the tendons protruded 27 in. from each end of the tubes.

The vertical edges of the slab forms had predetermined holes through which the

protruding prestressing wire ends were threaded to the outside of the forms when the tubes were placed in the forms.

First, the north-south direction wires were inserted followed by those in the east-west direction. The parabolic profile of the prestressing wires (Fig. 3) were accurately maintained and the tubes were correspondingly held in position.

To facilitate this positioning, chair inserts were placed under the wires at suitable locations. In addition, at the low

points, hooks were driven into the form plan to hold down the wires in position.

Thereafter, chucks were placed at the ends of the wires against the forms and the wires were held taut to prevent any possible rotation or translation of the wires. Then the plastic tubes were cut at the designated locations for strain-gage instrumentation. The wires were thoroughly degreased in those segments where strain gages were placed. They were then waterproofed and the outer layer liberally

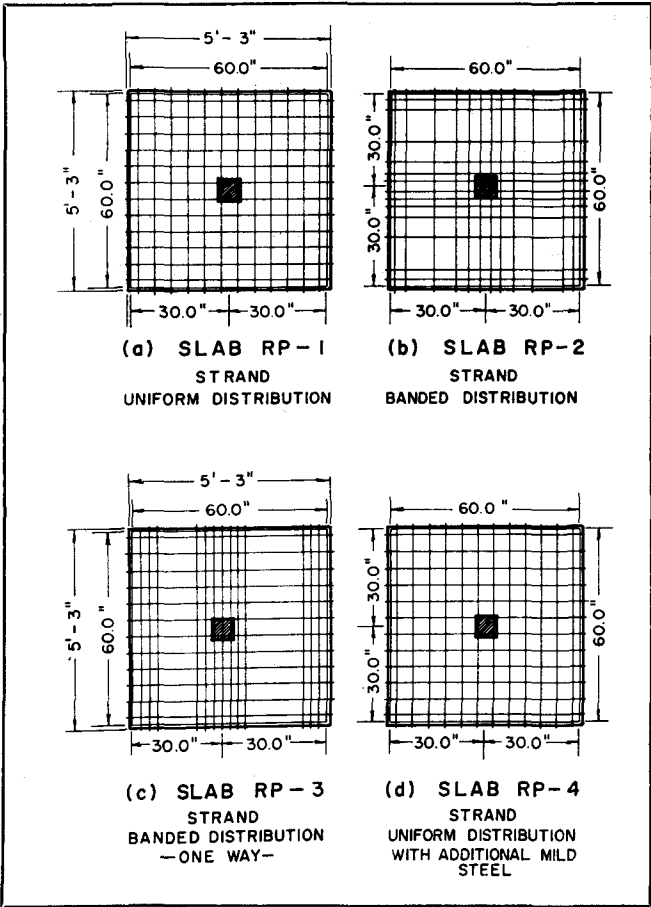


Fig. 4. Various plans showing distribution of prestressing steel reinforcement.

Table 2. Summary of test data of slab specimens.

Slab Designation	Ultimate Load P_u psi	First Cracking Load P_c psi	Age at Test days	Total Thickness (Avg) h in.	Effective Depth (Avg) d in.	Prestress Total A_{ps} Each Direction 12-wires (Dia. = .12") sq in. *	Prestress Steel Ratio percent	Normal Steel Total A_s Each Direction 12-bars (Dia. = .104") sq in. **	Normal Steel Ratio percent $f_y = 75$ ksi	Yield Strength of Prestress Wires f_{su} ksi
1	2	3	4	5	6	7	8	9	10	11
RP-1	10.716	7.433	9	1-5/16	1-1/32	.13572	.22	-	-	283
RP-2	11.091	7.232	9	1-5/16	1-1/32	.13572	.22	-	-	283
RP-3	10.748	6.671	9	1-5/16	1-1/32	.13572	.22	-	-	283
RP-4	11.728	7.024	9	1-5/16	1-1/32	.13572	.22	.10164	.1596	283

* A_{ps} per wire = 0.01131 sq in.

** A_s per bar = 0.00847 sq in.

greased. Each of these zones was then enveloped with an additional plastic sleeve of larger interior diameter to insure adequate movement of the wire during the prestressing operation.

The mild steel bar reinforcement mat (14 x 14 in.) was also inserted at the bottom in the center of the form for each slab at the column area to prevent possible shear failure. Two rows of mild steel reinforcement were also placed along the edges of the plates both at top and bottom.

Polyethylene wrapping was then placed around the prestressing chucks to protect them from concrete or moisture. The concrete was placed carefully in the forms and leveled to maintain an even thickness.

After 6 hr, each specimen as well as the control cylinders and the plain control beams were covered with wet burlap and cured continuously for 3 days. The polyethylene and chucks were then removed and the specimens stripped from the forms. Strain gages were subsequently placed at the top and bottom surfaces of the slabs.

Post-tensioning—The slabs were then prestressed after remounting the chucks to design stress level and readings taken. Alternate sides and alternate directions were taken at each individual tensioning of the various wires. Strain and deflection readings were recorded at this post-tensioning

stage prior to mounting the specimen on the testing machine.

Test setup—The prestressed slab was mounted on the test frame (Fig. 3). Since the uniform load was to be applied through a water-filled pressure bag, the water pressure was set at zero and the manometer reading was taken. The central load cell column (on a 4 x 4-in. area) was then installed at the center of the 5 ft 3 in. x 5 ft 3 in. area of the continuous plate. The edge supports were assembled to prevent the plate from uplift due to the upward load from the pressure bag.

The column was subsequently wedged so that the central upper deflection due to post-tensioning was reduced to zero. In this way the test specimen was ready for testing. Deflection bridges having deflection dial gages of 1/1000 in. accuracy were then placed on the tip of the specimens to measure deflections at the center and quarter-point of each panel.

As the hydraulic pressure of the system was applied, deflection and strain readings were taken at each load increment. In addition, the crack width was observed using high power microscopes of 1/2-mm accuracy. The loading was continued up to failure in all the specimens.

Prestressing strand—The strands in each orthogonal direction had a parabolic

profile as shown in Fig. 6. The initial prestressing force per 0.12 in. diameter wire varied from 1809.6 to 2035.8 lbs. The prestressing force at loading changed from 1747.5 to 1798.2 lbs as shown in Table 3. The corresponding stress before testing and after losses varied from 154.5 to 159.0 ksi.

Chucks were used as anchors for the strands which were tensioned using 10-ton hydraulic jacks. Strain measurements were taken both mechanically through strand elongation readings and through an electronic strain recording system.

Column loads (which were recorded by a load cell) were applied to reduce to zero the camber at center of the slab before application of the external test load. This load varied from 132.8 to 202.7 lbs. The calculated load varied from 80.53 to 123.01 lbs (Table 3). Camber due to this load was then superimposed on the camber already obtained.

Analysis of Test Results

Experimental deflection data

Table 4 gives the recorded deflection values at four major loading stages:

- (a) Basic load of 3.5 psi.
- (b) 50 percent of the ultimate load averaging about 5.5 psi.
- (c) First cracking load averaging 7.1 psi.
- (d) Loading stage prior to ultimate averaging 10.5 psi.

The basic load level used in this investigation was chosen as 3.5 psi to facilitate theoretical deflection computations by elastic procedures.

These procedures can be used easily up to the first cracking load. Thereafter, however, deflection computations must be adjusted using the cracked moment of inertia.

It should be stressed that in prestressed two-way slabs, even overload cracking should be preferably discouraged.

Theoretical deflection and camber calculations

Section 18.13 of ACI 318-71 stipulates^{1,2} that elastic plate theory, finite element or finite differences methods, equivalent frame method or the beam

Table 3. Summary of measured and calculated deflections in relation to applied prestressing forces.

Slab Designation	Initial Pre-stress f_{sp} psi	Final Pre-stress f_{se} psi	Initial Force P_o per Strand lb	Final Force P per Strand lb	Stress f_c in Concrete $\frac{P \times 12 \text{ str.}}{63 \times 1.3''}$ psi	Strand Force F per Unit Length of Plate Strand lb/in.	Necessary Column Wedging Force at Initial stage lb	Calc. Defl. Δ_c due to External Load $w=3.5$ psi in.	Meas. Defl. Δ_m due to External Load $w=3.5$ psi in.	Ratio $\frac{\Delta_c}{\Delta_m}$ -	Calcul. Camber Δ_c at Panel Center due to Pre-stress in.	Meas. Camber Δ_m at Panel Center due to Pre-stress in.
1	2	3	4	5	6*	7	8	9	10**	11	12	13
RP-1	160,000	154,509	1809.6	1747.5	256.04	332.85	103.38	.0310	.034	.911	.027	.031
RP-2	179,500	157,993	2030.2	1786.9	261.82	340.37	123.01	.0310	.036	.861	.027	.028
RP-3	170,500	156,994	1928.4	1775.6	260.16	338.21	118.55	.0311	.036	.864	.0269	.029
RP-4	180,000	158,992	2035.8	1798.2	263.47	342.51	80.53	.0302	.033	.915	.0266	.028

* Section demenstons based on plate length between chucks $\ell = 63''$ and average measured thickness $h = 1.3''$

** Compression of Neoprene pad ($\frac{1}{4}''$ thick 52 durometer) provided along edges compressed by estimated amount of .004" deducted from measured amount.

Table 4. Measured deflection data at various loading stages.

Slab Designation	Load Level												
	Basic Load * (w = 3.5 psi)			50 Percent Ultimate Load			First Cracking Load			Loading Stage Prior to Ultimate Load			Ult. Load
	Load psi	Panel Center defl. in.	Column Strip Center defl. in.	Load psi	Panel Cent. defl. in.	Column Strip Center defl. in.	Load psi	Panel Cent. defl. in.	Column Strip Center defl. in.	Load psi	Panel Cent. defl. in.	Column Strip Center defl. in.	psi
1	2	3	4	5	6	7	8	9	10	11	12	13	14
RP-1	3.5	.038	.032	5.358	.058	.052	7.433	.122	.101	9.964	.652	.472	10.716
RP-2	3.5	.040	.031	5.546	.062	.048	7.232	.083	.072	11.091	.632	.552	11.091
RP-3	3.5	.040	.035	5.374	.062	.051	6.671	.106	.101	10.062	.681	.560	10.748
RP-4	3.5	.037	.032	5.862	.064	.057	7.024	.084	.083	10.814	.318	.304	11.728
Avg.	3.5	.0388	.0325	5.535	.0615	.0505	7.064	.099	.094	10.481	.571	.471	11.070

*Note: 3.5 psi basic load is an average load level of approximately 50 percent of the first cracking load for the purpose of elastic deflection computations.

method may be used for flexural stress analysis. It also recommends that the span to thickness ratio should not exceed 48 for floors and 52 for roofs provided short-term and long-term deflection, camber, vibration frequency and amplitude are not objectionable.

Since prestressed slabs or plates are normally designed to be uncracked, the classical theory of small deflections of plates can be applied. While solutions are available for many simple conditions, continuity and stiffness effects of supporting columns or other members make the computations very tedious.

This paper will concentrate on the simplified equivalent frame method reduced to a beam method both for the deflection due to external load, and the opposing camber due to the parabolic prestressing strands under essentially working load conditions. A comparison will then be made with the elastic solution values from plate theory.

External load deflections

For a continuous flat plate supported by columns, the maximum deflection³

at the center of a square panel, using Poisson's ratio $\mu = 0.17$ and the flexural rigidity expression $D = Eh^3/[12 \times (1 - \mu^2)]$, is

$$\Delta = 0.11014 wl^4/Eh^3 \quad (1)$$

The coefficient $\alpha = 0.11014$ in Eq. (1) is subject to modification depending on the boundary conditions of the panels. A value of $\alpha = 0.149$ was reported for the boundary conditions given in Reference 4. For a floor system with support conditions similar to the current test condition, the deflection would be

$$\Delta = 0.0955 wl^4/Eh^3 \quad (2)$$

The above coefficient is obtained by finite differences when $\mu = 0.17$.

The coefficients of Eqs. (1) and (2) are therefore quite close. Eq. (2), however, approximates more closely the conditions of the test series in the present investigation.

The deflection equation by the beam theory⁵ for a uniformly loaded one-way plate restrained at one end and simply supported on the other is:

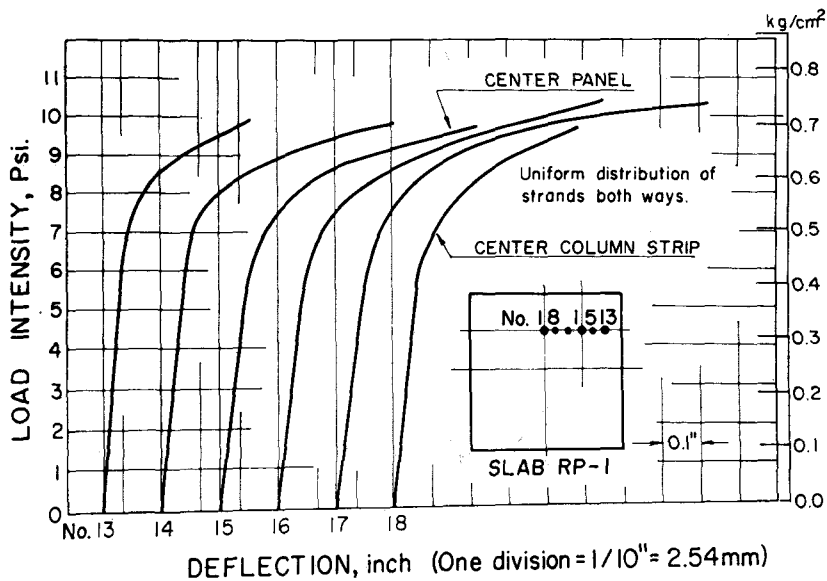


Fig. 5a. Load-deflection diagram for Plate RP-1.

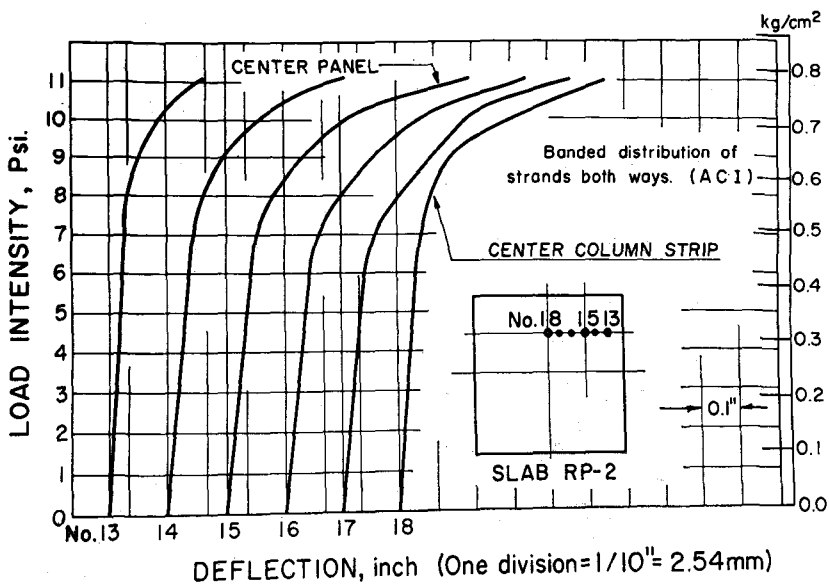


Fig. 5b. Load-deflection diagram for Plate RP-2.

$$EI \Delta = \frac{wlx^3}{16} - \frac{wl^3x}{48} - \frac{wx^4}{24} \quad (3)$$

For $x = l/2$, Eq. (3) reduces to

$$\Delta = 0.005208 wl^4/EI \quad (4)$$

Deflection computations due to a uniform external load stress intensity of 3.5 psi (approximately 50 percent of first cracking load) would follow the same procedure as in the equivalent frame method.⁶

The total deflection at the panel center would be the sum of the deflection Δ_{cx} at the center of the column strip in direction x and deflection Δ_{sy} at the center of the middle strip in direction y .

It should be noted that for the boundary conditions of the test specimens:

$$\Delta_{total} = \frac{1}{2}\Delta_{cx} + \Delta_{sy} = \frac{1}{2}\Delta_{cy} + \Delta_{sx} \quad (5)$$

As shown in the Appendix, the total calculated deflection resulting from Eqs. (4) and (5) is 0.031 in. (upwards). The total theoretical deflection from Eq. (2) is 0.0286 in. (upwards) which is lower than the equivalent frame-beam prediction.

Two-way prestressing camber

Evaluation of the camber due to the prestressing strands depends on the strand profile. The predetermined eccentricities that are required for the stress design would determine the load distribution of the strands across the span. Reference 7 gives a useful discussion on the preferable location of the strand profile (center of gravity of steel line) as necessitated by the pressure (center of gravity of concrete line).

Fig. 6 gives dimensional details of the parabolic profile of the strands, with eccentricities of $\frac{3}{8}$ in. at the central column center and at the point of curvature change in the span. On this basis, a load distribution due to prestressing gives a force of 332.85 lb per in. for Test Slab RP-1 (see the Appendix).

Computations lead to stress distri-

butions $q_A = 4.078$ psi (down), $q_B = 2.356$ psi (up), and $q_C = 8.79$ psi (up). By dividing the loads to partial loading, in order to simplify the calculations, a final prestressing camber of 0.027 in. (down) is found for Test Slab RP-1.

The net calculated deflection would then be the vector addition of the load deflection value of 0.031 in. (up) if the Eq. (5) result is used for deflection and the prestress camber value of 0.027 in. (down). In this case a net deflection of 0.004 in. (up) results giving a 1/7500 deflection to span ratio.

Thus, for all practical purposes, the slab can be considered to be plane when subjected to external load.

Discussion of Test Results

A comparison of the deflection plots in Fig. 5 (a,b,c,d) shows similar trends for both the centers of panels and centers of the column strips in all the four test series. Similar trends also occur well beyond the working load level.

The magnitude of deflection did not vary appreciably, regardless of the manner of distribution of the prestressing strands, namely, whether banded or uniform. However, close to the ultimate load, Series RP-4 which had additional mild steel reinforcement deflected considerably less.

This behavior is expected because the presence of additional mild steel reinforcement increases the stiffness of the slab. At 50 percent of the ultimate load, the deflection to span ratio at the panel centers was 0.0615/30 or 1/488 if no camber due to prestress is considered.

This deflection is well within normal ACI 318-71 Code limitations for serviceability requirements. The ratio of span to thickness is 23.1 which is also below the ACI 318-71, Section 18.13, stipulations.

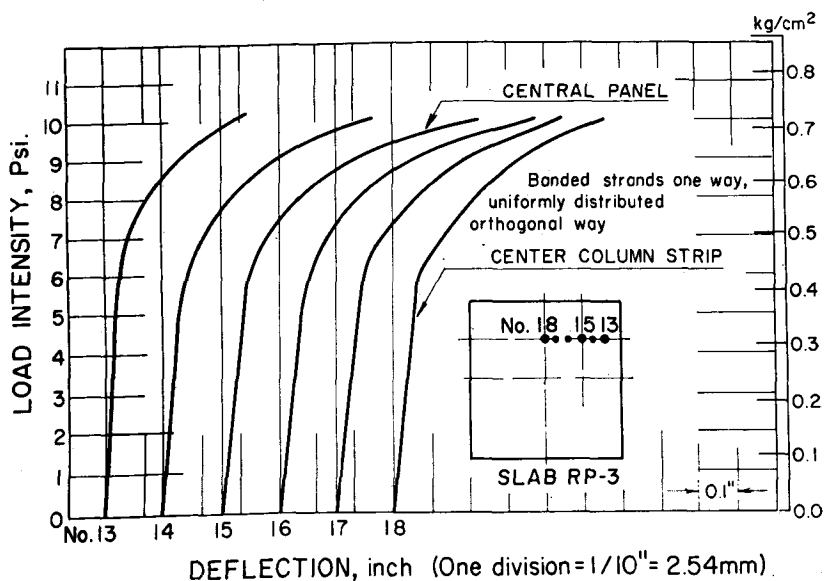


Fig. 5c. Load-deflection diagram for Plate RP-3.

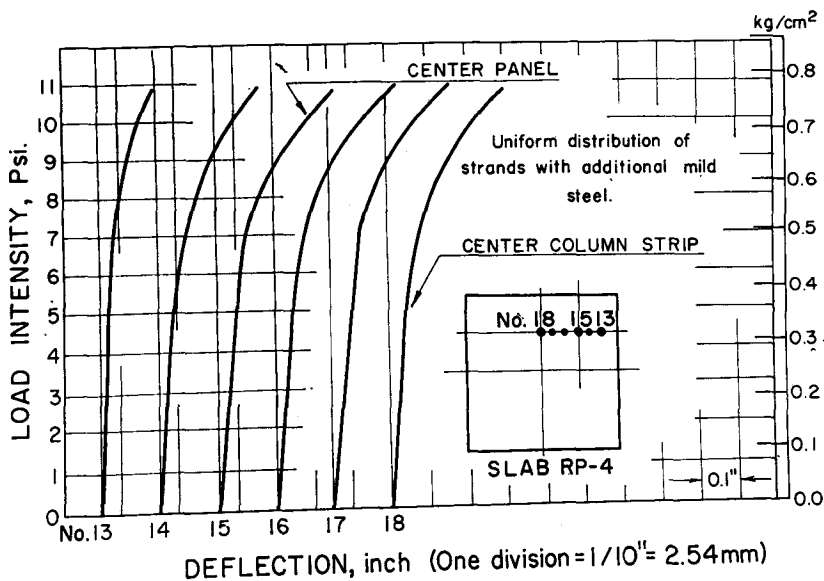


Fig. 5d. Load-deflection diagram for Plate RP-4.

Table 5. Panel deflection coefficients for various boundary conditions (square floor uniformly loaded).

Boundary Conditions	Deflection Coefficient α
Four edges built-in or interior panel of multi-panel beam slab system	0.00126*
Four edges simply supported	0.00406*
Interior panel of continuous multi-panel floor supported on columns	0.00581*
One-panel floor on columns	0.0257*
Four-panel floor on columns	0.0127**
Four panel floor, column supported at center and simply supported along edges	0.00819***

* Reference 3.

** Reference 4.

*** Navy-Chabrabarti tests.

Note: Deflection $\Delta = \alpha wL^4/D$

On the basis of relatively elastic behavior at working load levels of performance, the deflection at the center of a square panel with various boundary conditions is summarized in Table 5.

The limit state behavior of the test slabs at failure followed the normal yield line patterns expected in a four-panel plate having the applied boundary conditions.

Fig. 7 shows the yield line patterns in Test Specimen RP-4 at rupture after load release. This specimen failed at 11.73 psi uniform loading when it reached Loading Stage 10.

Fig. 8 shows Test Specimen RP-3 at Loading Stage 8 (9.14-psi uniform load). Dominant yield line cracks are seen at the tension sides of the midstrips. This specimen failed at Loading Stage 10 (10.75 psi uniform load).

An analysis of the limit state behavior of plates at failure will be the subject of a future paper.

Conclusions

1. A rational simplified method of calculating the deflection and camber of continuous two-way prestressed slabs is presented.

2. Prestressed slabs behave elastically up to the first cracking load. The elastic theory, simplified through an equivalent frame-beam deflection method, allows the engineer to design prestressed plates easily for serviceability loads.

3. Comparison of the experimental and theoretical results for camber and deflection shows good correlation.

4. Variation of distribution of the prestressing strands does not significantly alter the deflection behavior of the two-way slab or its magnitude for the same total prestressing steel percentage.

5. Addition of extra mild steel reinforcement to the prestressing steel, as in Series RP-4, decreased the deflection in the post-cracking loading stages by increasing the stiffness of the plate.

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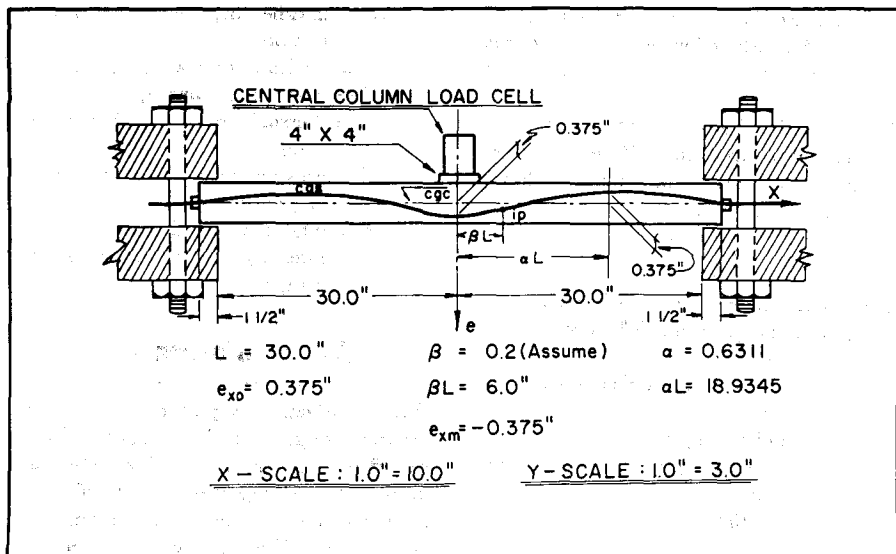


Fig. 6. Profile of prestressing tendons.

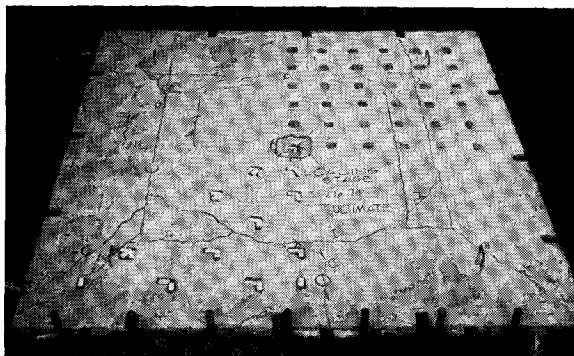


Fig. 7. Behavior of Plate RP-4 at ultimate load (Loading Stage 10).

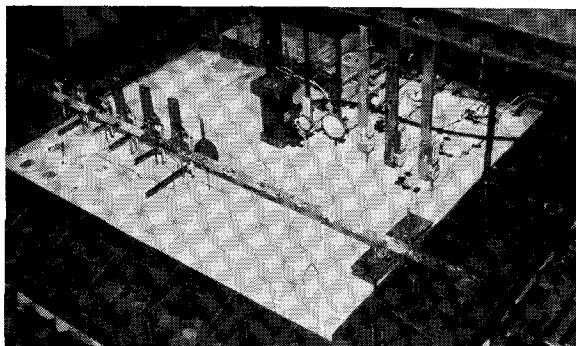


Fig. 8. Behavior of Plate RP-3 at 85 percent of ultimate load (Loading Stage 8).

proximate Calculations of Deflection of Two-Way Floor Systems," *ACI Journal*, Proceedings V. 72, No. 5, May, 1975, pp. 210-218.

7. Lin, T. Y., *Design of Prestressed Concrete Structures*, John Wiley and Sons, Inc., New York, 2nd Edition, 1963.

Notation

G_1 to G_6 = tendon force coefficients

Δ_{ex} = column strip deflection in x direction

Δ_{ey} = column strip deflection in y direction

Δ = deflection, in.

D = flexural rigidity of two-way plate

$$= \frac{Eh^3}{12(1 - \mu^2)}$$

$q_{A,B,C}$ = intensity of stress distribution due to prestress, psi

Δ_{sx} = middle strip deflection in x direction

Δ_{sy} = middle strip deflection in y direction

E = modulus of elasticity, ksi

I = moment of inertia, in.⁴

μ = Poisson's ratio = 0.17 for concrete used

F = prestressing force in one strand, lb per in.

h = total plate thickness, in.

w = intensity of external uniform load, psi

Acknowledgment

This investigation is part of a continuing research program on the behavior of two-way floor systems at the Concrete Laboratory of the Department of Civil Engineering—Rutgers, The State University of New Jersey. The program has been initiated and conducted under the direction of the first author. This paper is based in part on the PhD thesis of the second author.

APPENDIX—EXAMPLE FOR CALCULATING DEFLECTION

Example: Test Specimen RP-1

Beam-Equivalent Frame Method:

Data:

Panel span = 30×30 in.

Column strip = 15 in.

Middle strip = 15 in.

Average thickness, h = 1.3 in.

E = 4.315×10^6 psi

w per unit area = 3.5 psi

(a) External Load-Deflection

w per unit length = $3.5 \times 30 = 115$ lb per in.

$$I_{total} = bh^3/12 = 30(1.3)^3/12 = 5.49250 \text{ in.}^4$$

$$I_{ext.} = 15(1.3)^3/12 = 2.74625 \text{ in.}^4$$

$$I_{mid-strip} = 15(1.3)^3/12 = 2.74625 \text{ in.}^4$$

From Eq. (4):

$$\delta = 0.005208 w l^4 / EI$$

$$\begin{aligned} &= \frac{0.005208 \times 115(30)^4}{4.315 \times 10^6 \times 5.49250} \\ &= 0.02046 \text{ in.} \end{aligned}$$

Let K equal the effective stiffness of the equivalent column. In this case $K_1 = 0$. Assume that the stiffness coefficient α_1 from Reference 1 is zero and further assume that 50 percent moment is carried by the column strip and 50 percent of moment is carried by the middle strip. Therefore:

$$\begin{aligned} \Delta_{ex} &= 0.02046 (0.5) 5.49250 / 2.74625 \\ &= 0.02046 \text{ in.} \end{aligned}$$

$$\begin{aligned} \Delta_{ey} &= 0.02046 (0.5) 5.49250 / 2.74625 \\ &= 0.02046 \text{ in.} \end{aligned}$$

The total calculated deflection at the panel center due to the external load from the pressure bag is

$$\begin{aligned} \Delta_{ext} &= \frac{1}{2} \Delta_{ex} + \Delta_{sy} \\ &= 0.01023 + 0.02046 \\ &= 0.031 \text{ in. (upwards)} \end{aligned}$$

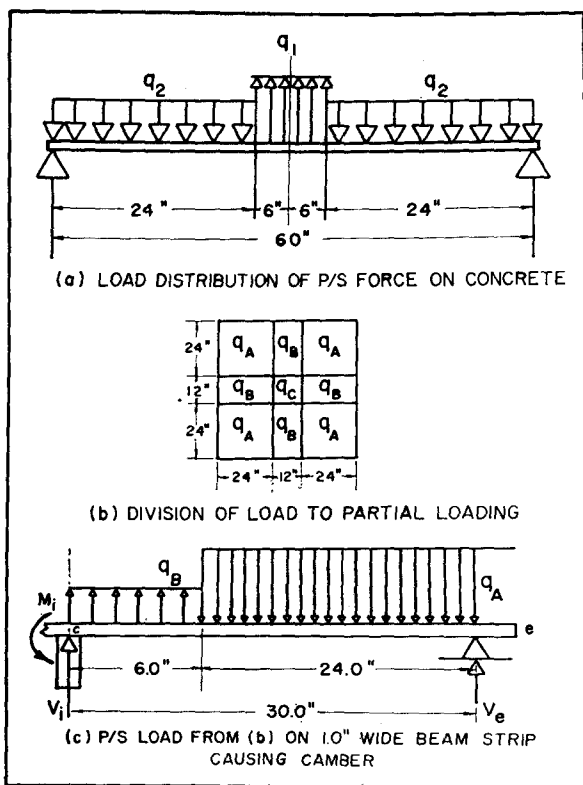


Fig. 9. Load distribution of prestressing tendons.

The measured deflection is 0.034 in. (upwards).

The calculated deflection by plate theory [Eq. (2)] is found from:

$$\delta = 0.0955 \frac{wl^4}{Eh^3}$$

$$= 0.0955 \left[\frac{3.5(30)^4}{4.315(10)^6(1.3)^3} \right]$$

$$= 0.0286 \text{ in. (upwards)}$$

(b) Two-way Prestressing Camber

From the strand's profile in Fig. 6, assume that the inflection point at the column region is 20 percent of span l . Therefore:

$$\beta = 0.20 \text{ and } \beta l = 6 \text{ in. for } l = 30 \text{ in.}$$

If the tendon profile is parabolic:

$$\alpha = 0.6311 \text{ giving } \alpha l = 18.9345 \text{ in.}$$

The force e on a string subjected to a uniform load can be defined as

$$e = G_1 x^2 + G_2 x + G_3 \quad \text{for } x \leq \beta l$$

$$e = G_1 x^2 + G_2 x + G_3 \quad \text{for } x \leq \beta l$$

For $e_{x=0} = 0.375$ and $e_{x=l} = 0.375$ in. used, the coefficients G give the following values:

$$G_1 = 0.00660183 \quad G_4 = 0.00306248$$

$$G_2 = 0 \quad G_5 = -0.11597168$$

$$G_3 = 0.375 \quad G_6 = 0.72292078$$

The load distribution of the prestressing force on the concrete would be as shown in Fig. 9a, where:

$$\frac{\partial^2 e}{\partial x^2} = 2FG_1 \text{ (upwards)}$$

$$q_2 = F \frac{\partial^2 e}{\partial x^2} = 2FG_2 \text{ (downwards)}$$

F = prestressing force, lb per in.

For Test Slab RP-1, $f_{se} = 154,509$ psi (measured)

$$\text{On 12 strands, } P = 154,509 \times 12 \times 0.01131 = 20,969.96 \text{ lbs}$$

$$\text{Unit stress } \sigma = \frac{P}{A} = \frac{20969.96}{63 \times 1.3} = 256.04 \text{ psi}$$

$$F = \sigma h = 256.04 \times 1.3 = 332.85 \text{ lb per in.}$$

$$q_1 = 2FG_1 = 2 \times 332.85 \times 0.00660183 = 4.395 \text{ psi (upwards)}$$

$$q_2 = 2FG_2 = -2 \times 332.85 \times 0.00306229 = -2.039 \text{ psi (downwards)}$$

Dividing the loads to partial loading in order to simplify the calculations as in Fig. 9b:

$$q_A = q_2 + q_3 = 2.039 + 2.039 = 4.078 \text{ psi (downwards)}$$

$$q_B = q_1 + q_2 = 4.395 - 2.039 = 2.356 \text{ psi (upwards)}$$

$$q_C = q_1 + q_1 = 4.395 \times 2 = 8.79 \text{ psi (upwards)}$$

To calculate the prestress camber due to the partial loadings of Fig. 9b for a beam strip 1-in. wide, assume the effect of prestressing strand forces longitudinally as evenly distributed over the whole 30-in. wide section.

As in the case of the approximated deflection computations by the equivalent frame-beam approach, assume both the column strip and the middle strip can simulate a one-way plate restrained at the interior end *i* and simply supported at the exterior end *e* in Fig. 9c.

The deflection or camber expression applicable for such a case is

$$\Delta = 0.00528 w_p l^4 / EI$$

where w_p is the unit intensity of distribu-

tion of prestressing force (psi) and EI is the rigidity of 1-in. wide strip.

For Test Slab RP-1:

$$E_{ci} = 3.600 \times 10^6 \text{ psi}$$

$$I = 1 (1.3)^3 / 12 = 0.1831 \text{ in.}^4$$

$$EI = 0.659 \times 10^6 \text{ lb-in.}^2$$

Because the direction of prestressing intensity is upwards at the column region (q_B) and downwards at the span (q_A), the average stress is:

$$w_p = (4.078 \times 24 - 2.356 \times 6) / 30 = 2.7912 \text{ psi}$$

$$\begin{aligned} \text{Camber } \Delta'_{cy} &= \frac{0.005208 \times 2.7912(30)^4}{0.659 \times 10^6} \\ &= 0.0179 \text{ in. (downwards)} \end{aligned}$$

Similarly, $\Delta'_{sx} = 0.0179 \text{ in. (downwards)}$.

Therefore,

the total calculated at the panel center is

$$\begin{aligned} \Delta' &= \frac{1}{2} \Delta'_{cy} + \Delta'_{sx} \\ &= 0.00895 + 0.0179 \\ &= 0.027 \text{ in. (downwards)} \end{aligned}$$

(c) Net Deflection or Camber

From Part (a), the calculated deflection is 0.031 in. (upwards).

From Part (b), the calculated camber is 0.027 in. (downwards).

The net calculated deflection when external load is applied is 0.004 in. (upwards).

It should be noted that the direction of the external load in the test slabs was upwards.

Discussion of this paper is invited. Please forward your discussion to PCI Headquarters by August 1, 1976.

Reviews of Current Publications

Structural Fire Resistance of Concrete

Eduardo Salse and Tung D. Lin

A method is presented for calculating the structural fire endurance of concrete members under various conditions of axial and bending restraints. Included is the calculated temperature distribution during a fire within concrete sections. Practical examples are given. Conclusions were that:

1. Temperature distributions can be calculated for various cross sections and aggregate types, and from these data and information on temperature-strength relations, the material strength can be obtained at any time during a fire.

2. Equilibrium conditions at collapse, considering bending and axial restraint simultaneously, were obtained and show that time of collapse depends on the axial restraining force, its line of action, and member deflections.

3. Axial restraining forces increase fire resistance.

4. For interior bays, fire resistance is independent of the restraining force locations.

5. Equilibrium conditions are independent of deflections whenever axial restraining forces are neglected.

(*Journal of the Structural Division*, ASCE, V. 102, No. ST1, Proceedings Paper 11835, January, 1976, pp. 51-63)

Epoxy Adhesives in Prestressed and Precast Concrete Bridge Construction

Raymond J. Schutz

The use of epoxy resin based adhesives in two different types of precast concrete bridge construction are described. The bridges covered are the conventional segmental bridges and a new system of precast concrete bridge deck construction. Construction procedures are described, and the physical properties of

the different types of adhesives required are reviewed.

(*ACI Journal*, Proceedings, V. 73, No. 3, March 1976, pp. 155-159)

Concrete Cylinder Structures Under Hydro-Static Loading

Harvey H. Haynes and Bjorn A. Nordby

Several design equations are presented to predict implosion of thin-walled and thick-walled concrete cylinder structures under hydrostatic loading.

Of significance is the development of a modified Donnell equation to analyze moderately long, thin-walled cylinders for implosion. Most concrete structures for offshore oil production platforms fall in this category.

Donnell's equation is modified by an empirical plasticity-reduction factor which varies from 1.0 to 0.10 for cylinder wall stress at implosion that vary from $0.4f_c'$ to $1.1f_c'$ respectively.

(*ACI Journal*, Proceedings V. 73, No. 2, February 1976, pp. 87-96)

Design of Precast Concrete

Robert A. Hartland

This compact book, which is the first in a series dealing with various practical aspects of design and materials, is aimed at the young engineer although it has much useful information for more experienced designers.

The text covers framed structures in precast concrete, large panel construction, loadbearing and non-loadbearing cladding panels, and practical examples of design and in detail for a wide range of structures. These designs are in accordance with the British Code of Practice (CP 110).

(Halsted Press, a Division of John Wiley & Sons, Inc., 605 Third Avenue, New York, N.Y. 10016, 1976, 148 pp., \$12.50)

Reference Cards

KEYWORDS: abnormal loads; bracing; connections; continuity; creep; deflection; design loads; design (structural); diaphragms; differential settlement; expansion joints; flexural strength; floors; foundations; high-rise buildings; hollow-core panels; horizontal joints; joints; loadbearing walls; load distribution; load transfer; long-term effects; PCI Committee report; precast concrete; prestressed concrete; progressive collapse; reinforcement; restraint; roofs; sandwich panels; shear connections; shear keys; shear walls; shrinkage; stability; stiffness; ties; tolerances; transfer girders; vertical joints; volume changes; walls.

ABSTRACT: Presents design guidelines for precast bearing wall buildings subject to abnormal loadings. Emphasizes horizontal, vertical, and peripheral ties sufficient to provide interaction between all building elements. Empirical design approaches, based on various reports, are suggested and some examples are provided in the Appendix. The following chapters are covered: (1) Object and range of application; (2) Design loads; (3) Stability; (4) Foundations; (5) Structural systems; (6) Floors and roofs; (7) Loadbearing walls; (8) Shear walls; and (9) Joinery.

An Appendix section includes some empirical suggestions for designing transfer girders and some methods for determining the load capacity of horizontal joints.

REFERENCE: PCI Committee on Precast Concrete Bearing Wall Buildings, "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads," PCI JOURNAL, V. 21, No. 2, March-April 1976, pp. 18-51.

KEYWORDS: design (structural); economy; moment; prestressed concrete; prestressing steel; strain compatibility; strand; strength; T-beams.

ABSTRACT: The purpose of this article is to show that strand savings (in terms of additional resisting moment) are possible when one uses the more accurate strain compatibility method allowed by the ACI Code.

A design example compares the results using Eq. (18-3) of the ACI Code and a strain compatibility method. This latter method utilizes charts in the PCI Design Handbook.

REFERENCE: Aswad, Alex, "Strand Savings Using Strain Compatibility in Strength Design," PCI JOURNAL, V. 21, No. 2, March-April, 1976, pp. 78-81.

KEYWORDS: Bangkok, Thailand; driving, foundation; pile splice; piles; precast concrete; splicing.

ABSTRACT: The author, who was a consultant on this job, describes a splicing method which salvaged a sure loss while furnishing piles of adequate capacity for the foundation of a multilevel garage in Bangkok, Thailand.

REFERENCE: Kramer, C. R., "Splicing Technique Solves Major Precast Pile Salvaging Operation," PCI JOURNAL, V. 21, No. 2, March-April, 1976, pp. 82-85.

KEYWORDS: brackets; connections; corbels; cracking; design recommendations; flexural strength; lightweight concrete; normal weight concrete; precast concrete; reinforced concrete; reinforcement; research; shear-friction; shear strength; stirrups; tests; ultimate strength.

ABSTRACT: An experimental study is reported of the behavior of reinforced concrete corbels subjected to both vertical and horizontal loads.

Twenty-eight corbel specimens were tested, of which twenty-six contained horizontal stirrup reinforcement. The variables included in the study were the shear span to effective depth ratio, the ratio of the vertical load to the horizontal load, the amounts of main tensile and stirrup reinforcement, and the type of aggregate.

Criteria for the design of horizontal stirrup reinforcement are developed. It is shown that, subject to provision of the recommended amount of stirrup reinforcement, the useful ultimate strength of corbels can be taken to be the lesser of

(a) The shear strength of the corbel-column interface, calculated using either the shear-friction provisions of Section 11.15 of ACI 318-71 or the modified shear-friction equation, and;

(b) The vertical load corresponding to the development of the flexural ultimate strength of the corbel-column interface.

In the next issue a follow-up paper will propose specific code provisions for designing corbels together with numerical examples.

REFERENCE: Mattock, Alan H., Chen, K. C., and Soongswang, K., "The Behavior of Reinforced Concrete Corbels," PCI JOURNAL, V. 21, No. 2, March-April, 1976, pp. 52-77.

KEYWORDS: analysis; banded tendons; camber; cracking; deflection; flat plates; models; post-tensioned construction; prestressed concrete; research; slabs; two-way action; uniform tendons.

ABSTRACT: Reports the results of a theoretical and experimental investigation into the deflection behavior of four 60 x 60-in. four-panel continuous prestressed flat plate models.

Each panel of the slab specimens is 30 x 30 in. square and is 1-5/16 in. thick. The plates were subjected to a uniformly distributed load with post-tensioning applied in two directions.

The strand was draped parabolically and was distributed both uniformly and banded. To control cracking, mild steel reinforcement was added in one of the slab specimens. A rational method, based on an equivalent beam-frame analogy, was developed to calculate deflection and camber in two-way prestressed flat plates.

The Appendix section includes a sample calculation for determining the deflection and camber in one of the test slabs. The experimental results show good correlation with the theory through the elastic range.

The subject of this paper is confined to plate behavior up to the first cracking load. A future paper will deal with the post-cracking behavior of plates.

REFERENCE: Nawy, Edward G., and Chakrabarti, Pinaki, "Deflection of Prestressed Concrete Flat Plates," PCI JOURNAL, V. 21, No. 2, March-April, 1976, pp. 86-102.

Industry News

T. Y. Lin Symposium on Prestressed Concrete

A high-powered one-day symposium on prestressed concrete has been planned for Saturday, June 5, 1976, at the University of California at Berkeley to honor the retirement of PCI Member Professor **T. Y. Lin** from the University.

The theme for the symposium is "Prestressed Concrete—Past, Present and Future" and will include eight papers to be presented by eminently qualified professionals in the prestressed concrete field. Following the symposium, a retirement banquet will be held in the evening. Displayed at the symposium will be an exhibit of Professor Lin's major projects and other works.

The following papers will be presented:

1. "Historical Perspective" by **David P. Billington**, professor of civil engineering, Princeton University, Princeton, New Jersey.
2. "Influence of Research" by **Alan H. Mattock**, professor of civil engineering, University of Washington, Seattle, Washington.
3. "Bridges" by **John J. Kozak**, chief, Division of Structures and Engineering Services, Department of Transportation, State of California.
4. "Buildings" by **Alfred A. Yee**, structural engineer, Honolulu, Hawaii.
5. "Ocean Structures" by **Ben C. Gerwick, Jr.**, professor of civil engineering University of California, Berkeley, California.
6. "Nuclear Structures" by **D. W.**

Halligan, chief civil engineer, Bechtel Power Corporation, Gaithersburg, Maryland.

7. "Manufacture and Production" by **W. Burr Bennett, Jr.**, executive director, Prestressed Concrete Institute, Chicago, Illinois.
8. "The Future—A Long Look Ahead" by **T. Y. Lin**, professor of civil engineering, University of California, Berkeley, California.

Professor T. Y. Lin, who has been faculty member of the Department of Civil Engineering of the University of California, Berkeley, since 1946, will be retiring from University service at the end of the present academic year. Professor Lin, who also is founder and president of his own consulting firm, has had a long and distinguished career. He has been closely identified with the development of prestressed concrete design and construction in the world.

The Symposium program will be held at the Physical Sciences Lecture Hall, University of California, Berkeley. Registration Fee for the symposium will be \$25 and will include attendance at the Symposium, lunch and a copy of the printed "Proceedings" to be mailed at a future date to all participants. The total cost of the banquet will be \$15.00 per person.

The "Proceedings" of the Symposium will be published in a special issue of the PCI JOURNAL.

Further details regarding the program and reservations may be obtained by writing to Mrs. R. Burde, secretary, T. Y. Lin Program Committee, 761 Davis Hall, University of California, Berkeley, California 94720.

Holland Appointed PCI President

Eugene P. Holland has been named President of the Prestressed Concrete Institute, filling the vacancy created by the resignation of **Edward Schechter** of Stressteel Corporation.

Mr. Holland, co-founder and president of the Chicago-based structural engineering firm of Wiesinger-Holland Ltd., has served the PCI for several years. During this time he has been active in technical committee work and in administrative affairs as a member of the PCI Board of Directors and most recently as chairman of the Technical Activities Committee. Currently, he is chairman of the important ACI Building Code Committee.

Other current Officers of the PCI are **Leonard M. Perlmutter** (Prestressed Concrete of Colorado, Inc.), Vice President-President Elect; and **Gerd H. Marohn** (Arnold Stone Company), Secretary-Treasurer.

Mr. Holland's firm, Wiesinger-Holland Ltd., is engaged in the design of building structures with an annual construction volume of more than 100 million dollars, achieving a total of over one billion dollars since its founding in 1965. He is also a lecturer with the Architectural Department, University of Illinois, Chicago Circle Campus.

Mr. Holland is the originator of numerous structural innovations to improve the construction of buildings and has authored many notable technical and professionally-oriented papers including co-authorship of the book "Housing" to be published this summer by Wiley Interscience, Division of John Wiley & Sons, Inc.

The Prestressed Concrete Institute will continue to represent the prestressed concrete industry in its



Eugene P. Holland
(New PCI President)

broadest sense through its strong technical and research committee structure together with the current strong involvement of the professional members in PCI. Likewise, the PCI JOURNAL will continue to be the technical voice of the entire prestressing industry. This means that articles on post-tensioning technology will continue to be published in the PCI JOURNAL. (See also President's Message in this issue for elaboration of this editorial policy.)

Post-Tensioning Institute Formed

A new Post-Tensioning Institute (PTI), representing most of the post-tensioning companies in the United States and Canada, has been incorporated as a non-profit corporation in the State of Illinois.

The executive director of PTI is **Clifford L. Freyermuth** who was the director of the former Post-Tensioning Division of the Prestressed Concrete Institute.

The offices of PTI will be at 1701 Lake Avenue, Glenview, Illinois 60025.

Readers' Comment

Recommendations for estimating prestress losses*

Reported by PCI Committee on Prestress Losses

Comments by William L. Gamble and A. I. Fadl, William J. Gregg, Daniel P. Jenny, James J. Mallett, Maher K. Tadros and Amin Ghali, Edwin B. Workman, and Committee.

William L. Gamble and A. I. Fadl*

The report by the PCI Committee on Prestress Losses is a welcome addition to the literature on long-term behavior of prestressed concrete. The writers wish to make some specific comments about the material presented, and also some more general comments on the whole philosophy of loss calculations.

First, we feel the environmental conditions, and especially relative humidity, should be taken into account in evaluating losses in steel stress with time. There is surely a significant difference between the losses in an enclosed, heated building and an unenclosed structure. A long period of time for construction would help reduce the differences between the two cases, but the low indoor relative humidities found in the winter in most of the United States must increase the shrinkage and possibly the creep to values very much higher than in unenclosed, unheated structures. This is especially true for the thinner members often used in precast, pretensioned building members.

The relationships between relative ultimate creep and shrinkage and volume/surface ratio presented in Figs. 3 and 8

and Tables 3 and 6 are based on tests under constant humidity and temperature conditions. There is no evidence to support the validity of these relationships for a concrete subjected to the annual variations in weather, and there is evidence³⁷ to suggest that if there is a size effect outdoors, it is different and probably less important, at least in a midwestern climate.

The approximation that $f_{si} = 0.90 f_t$ [Eq. (18)] is a poor generalization, except for the most preliminary calculations. The warning about heavily and lightly prestressed members is not meaningful, since what a bridge designer might consider to be only moderately prestressed might appear as quite heavily prestressed to the person concerned with apartment building slabs.

In this era of computers and minicalculators, there is no excuse for using a simple approximation when the correct solution is not very hard, either. For a weightless member, the steel stress immediately after detensioning can be calculated by the expression:

$$f_{si} = \frac{f_t}{1 + n A_s \left(\frac{1}{A_o} + \frac{e^2}{I_o} \right)}$$

All symbols have the meanings defined

*PCI JOURNAL, V. 20, No. 4, July-August 1975, pp. 43-75.

†Professor of Civil Engineering and Research Assistant in Civil Engineering, respectively, University of Illinois at Urbana-Champaign.

in the Committee report except that f_t should be reduced because of relaxation between tensioning and detensioning, and with the additional term:

$$n = E_s/E_{ci} = \text{modular ratio.}$$

The actual steel stress is slightly higher than this value, because of the dead load moment, and the additional stress can be approximated as

$$\Delta f_s = n M' e / I_s$$

This is of course merely the dead load stress in the concrete at the level of the center of gravity of the steel, times the modular ratio.

The writers would like to make a few more general comments on losses in prestressed members. As is stated in the paper, the losses make no difference to the flexural strength of a member with bonded tendons. Since the problem is consequently one of serviceability rather than safety, the material properties assumed in the analysis of losses do not necessarily have to be those assumed in the strength part of the design. There are two outstanding examples.

In precast, pretensioned construction, the specified 28-day cylinder strength almost never controls the mix design. The mix is designed so that the specified strength at detensioning, f_{ci} , is reached in the shortest possible time. The real 28-day strength of the cylinders is then generally considerably higher than the specified value. Values of 6000 to 6500 psi and higher are common for members with specified $f'_c = 5000$ psi. We wish to suggest that the long-term behavior of the member should be evaluated using this higher concrete strength.

A second area in which there seems to be a difference between the material specified and the material supplied is the prestressing strand. The relaxation of strand under constant strain conditions can be reasonably predicted using Eq. (16) [or Eq. (17) for low-relaxation steel], which came from Reference 13. The total relaxation loss is a function of the ratio of initial stress to yield stress, but not in a linear manner. A relatively small increase in f_y above the design value can lead to a reasonably important reduction in the relaxation component of the loss. If $f_{ci} =$

189 ksi, increasing f_y by 15 ksi above the design value results in a 6-ksi reduction in the computed 20-year relaxation value.

In Eq. (16), f_y was defined in Reference 13 as the stress at an offset strain of 0.001. This is not necessarily the same stress as one finds at a total strain of 0.01, f_{py} , or at 0.85 f_{pu} ; the differences may be important.

While the specified $f_{py} = 0.85 f_{pu} = 230$ ksi for Grade 270 strand and 212 ksi for Grade 250 strand, the actual values appear to be considerably higher than this. Samples of strand tested for use in various laboratory projects and samples obtained from fabrication of actual pretensioned girders have invariably been stronger than these values, with a strongest single value (duplicate specimens)³⁸ of $f_{pu} = 298$ ksi and $f_y = 258$ ksi for a 7/16 in., Grade 250 strand!

Data from acceptance tests on strands from two steel suppliers for delivery to two beam fabricators was also analyzed. The results of these tests are summarized in Table A, with f_y determined at the 0.001 offset strain.

If the values in this table are representative of current strand production, it would appear that $f_y = 245$ ksi would be a reasonable value for use in evaluating the relaxation loss in Grade 270 strand. For these samples, f_{py} at 0.01 strain averaged 10 to 12 ksi higher than the f_y values listed.

With $f_y = 245$ ksi, pure relaxation at 20 or 30 years is computed to be about

Table A. Measured yield stresses for 7-wire strand.

	Manufacturer #1			#2
	7/16	7/16	1/2	7/16
Size, in.	7/16	7/16	1/2	7/16
Grade	250	270	270	270
No. Tests	7	7	27	7
Avg. f_y ,** ksi	238.7	250.7	246.3	252.8
Minimum, ksi	237	246	235*	247
Maximum, ksi	241	257	256	256

* Only value less than 241 ksi.

** f_y at 0.001 offset strain.

22 ksi, rather than the more widely quoted 28 ksi. The relaxation in a beam is of course smaller than this, but the difference between the two cases would remain about the same. The 6-ksi difference in steel stress is of some importance as long as the design is based on some limiting concrete stress in tension, though this difference is not important in so far as the actual behavior of the member is concerned.

It might be noted that the general method of computing losses must be very carefully handled to insure that equilibrium between steel and concrete forces is restored at the end of each time interval. If this is not done, there will be a cumulative error in equilibrium.

References

37. Mossiessian, V., and Gamble, W. L., "Time-Dependent Behavior of Non-composite and Composite Prestressed Concrete Structures Under Field and Laboratory Conditions," Civil Engineering Studies, Structural Research Series No. 385, University of Illinois, Urbana, May, 1972, 517 p.
38. Houdeshell, D. M., Anderson, T. C., and Gamble, W. L., "Field Investigation of a Prestressed Concrete Highway Bridge Located in Douglas County, Illinois," Civil Engineering Studies, Structural Research Series No. 375, University of Illinois, Urbana, February, 1972, 138 p.

William J. Gregg*

The PCI Committee on Prestress Losses is to be commended for its efforts in presenting this up-to-date information in a clear and concise manner.

The following comments are offered for consideration by the Committee:

*Chief Design Engineer, Con-Force Products, Ltd., Calgary, Alberta.

1. For convenient incorporation of this data into computer programs, it is preferable that suitable equations be developed that would be equivalent to the tabular data presented in the report. Such equations could be polynomial or logarithmic in derivation, and should be continuous, if possible, within the practical range of variables encountered within the industry. Figs. A, B, and C, shown herein,

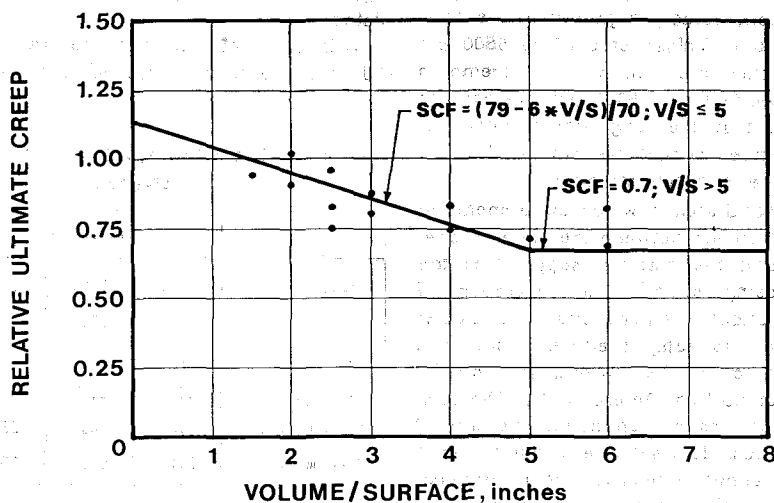


Fig. A. Relative ultimate creep versus volume to surface ratio. (See Fig. 3 in Committee report.)

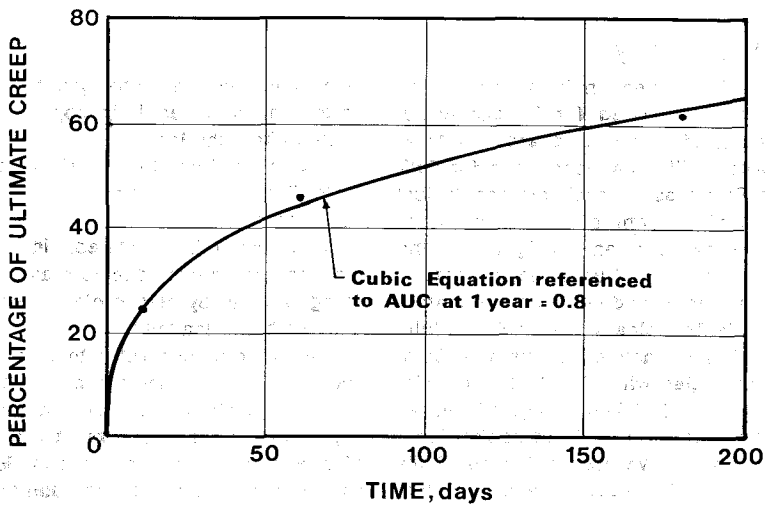


Fig. B. Percentage of ultimate creep versus time. (See Fig. 5 in Committee report.)

have been reproduced from the Committee report, and show suggested equations which are essentially equivalent to the tabular data in Tables 3, 5, and 6.

2. When consideration is directed to the scatter of the data from which equations for *UCR* and *USH* were derived, it may be appropriate that the relationships for *AUC* and *AUS* be combined into one equation, in the interest of simplification.

3. The Committee report is a comprehensive study of a complicated subject, and thus provides a valuable technical aid to the industry. Losses calculated in accordance with the general method as outlined in the report should be interpreted as estimated values only, and may require that the results be adjusted to conform to local experience within the industry.

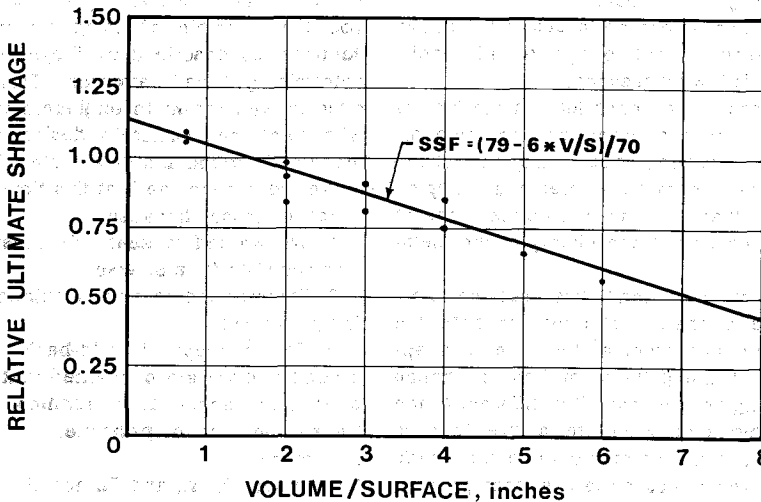


Fig. C. Relative ultimate shrinkage versus volume to surface ratio. (See Fig. 8 in Committee report.)

Daniel P. Jenny*

The PCI Committee on Prestress Losses is to be commended for its successful completion of a long and arduous task. Its report on "Recommendations for Estimating Prestress Losses" represents just one part of its many contributions to advanced knowledge and insight into the difficult, sometimes frustrating, aspects of losses in prestressed concrete members.

As PCI's Technical Director, I was fully aware of the many questions the Committee struggled with. One had to do with the concept of "elastic regain" or, as many popularly call it, "add-back." From time to time I have been asked for PCI's position on this question, and now that the Committee's report on losses is published, I have been asked why the concept is not included in the report. Perhaps the following comments may be of help to others:

The "add-back" concept has led to much heated debate over the years during the preparation of the Committee report on Prestress Losses and the current recommendations that appear in the *PCI Design Handbook*. I fear there are still two schools of thought on this question, but currently the Institute position on losses is presented in the AASHTO Interim Specifications—Bridges 1975, in the Committee report published in the July-August 1975 *PCI JOURNAL*, and in the *PCI Design Handbook*.

The discussions on losses in the above documents do not recognize "add-back" for the following reasons:

1. Losses are dependent on the history of permanent stresses on the member. These continual, but changing, stress loads on the member create a complex pattern that leads to a predicted loss at any given time in the history of the member.

2. A transient load (live load) certainly creates a change of stress in both the concrete and steel at the time it is applied but, being transient, has no chance of changing the loss. Put differently, the loss immediately before a live load is applied, is a predictable value that leads to a given stress pattern at that point in

time; the live load changes the stress pattern as soon as it is applied, but it cannot affect the loss.

3. Quoting from the *Prestressed Concrete Designer's Handbook*,³⁹ by Abeles and Turner:

"It is sometimes stated, incorrectly, that loss at transfer due to elastic shortening is offset by elastic elongation when the member is loaded."

The authors then refer to loss formulas and other arguments and conclude:

"It is therefore immaterial whether lifting of the member occurs at the instant of prestressing or whether it is delayed, as the combined elongations due to bending cannot affect the magnitude of the loss due to elastic shortening."

4. If the arguments in favor of "add-back" have any validity at all, and it can only occur because of transfer of stresses from the concrete to the steel through bond, then why not argue in the same fashion for receiving some amount of "prestressing" in a normally reinforced beam when live load is applied, and thus anticipate greater load-carrying ability in the beam?

The question of "add-back" or elastic regain was brought into focus by Dr. Anderson's President's Message in the July-August, 1971, *PCI JOURNAL*. In the reader response in the March-April, 1972, *PCI JOURNAL*, I especially recommend to you the principle of using transformed sections, as described by Freyermuth, in determining "real" stresses. This principle is well-known to engineers familiar with reinforced concrete design and is technically sound and defensible.

We did not include it at this time in our report on losses because:

1. We wanted to keep the question of loss separate from stresses.

2. Transformed section calculations are fairly involved.

3. The concept of "add-back" is still around and we are concerned that someone might apply it in addition to the transformed section principle.

Reference

39. Abeles, P. W., and Turner, F. H., *Prestressed Concrete Designer's Handbook*, Concrete Publications Ltd., London, England, 1962.

*Technical Director, Prestressed Concrete Institute, Chicago, Illinois.

James J. Mallett*

The Committee is to be congratulated for its recommendations which are significantly more accurate than the old method in the *PCI Design Handbook*. The Committee's recommendations will, in most cases, save prestressing strand.

This discussion is based on observations made of the new criteria since April 1975. Not concerned with individual relationships, a pragmatic approach was taken to discover how well the procedure would work when included in a system and applied to actual structures.

A large percentage of prestress design is controlled by allowable bottom tensile stresses. The engineer, to confidently use the higher tensile stresses sanctioned by the ACI Building Code (ACI 318-71), must have a reliable method for computing losses.

Unfortunately, except for rare occasions when the structures are instrumented, he will never know the actual level of tensile stresses. If inspections confirm the anticipated cambers at erection, it is generally agreed that the assumed losses were correct.

Not infrequently, however, structures erected with the proper camber have,

years later, shown excessive deflections. Time-dependent losses computed as outlined by the Committee's recommendations should, to a large degree, eliminate such problems in the future.

While the recommendations define the time-dependent losses very adequately, one additional factor must be included for accurate camber/deflection predictions. For the examples shown, the inelastic creep influences were calculated, not estimated. To duplicate the predicted cambers, growth factors ranging from 0.6 to 1.6 are required. Obviously, the use of growth factors can lead to unpleasant surprises for the unwary engineer.

A camber growth study involving fourteen 8 ft wide by 36 in. deep single tees is summarized in Fig. D. At 6 months the average camber was 1.47 in. as compared to a prediction of 1.50 in. Production scheduling necessitated varying the de-tensioning age from 14 to over 60 hours and both moist and accelerated curing methods were used. Release strengths

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A Subsidiary of Westinghouse, Pensacola, Florida.

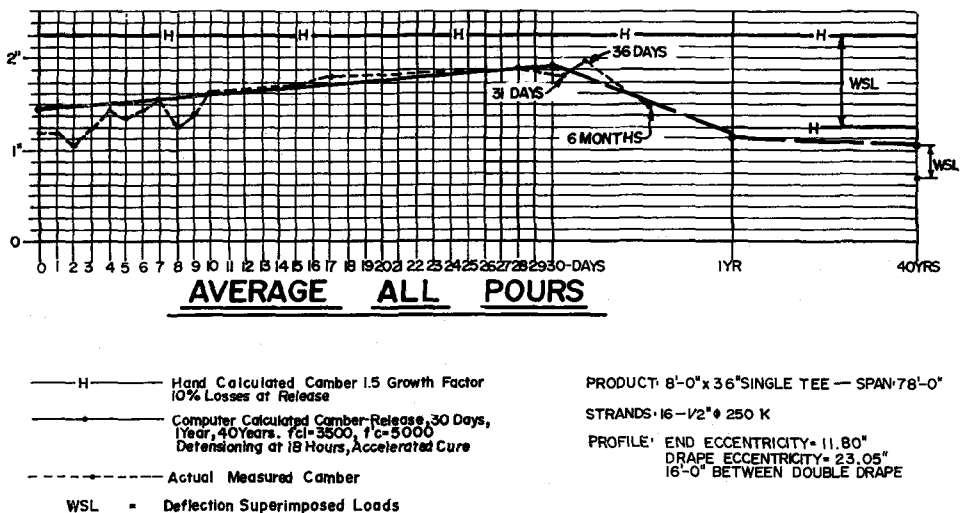
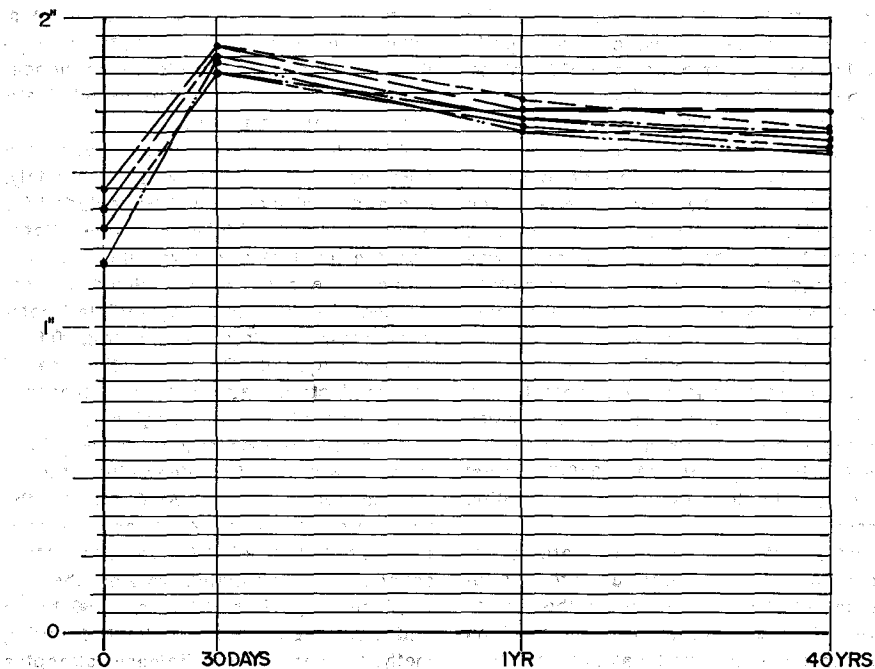


Fig. D. Comparison of hand-calculated camber, computer-calculated camber, and actual measured camber for prestressed normal weight concrete single tee.



LEGEND

SYMBOL	RELEASE	DESIGN	CURE
————	3600	5000	Accelerated
-----	4000	5000	do
-----	4500	5000	do
-----	3600	6000	do
-----	4500	6000	do
-----	5000	7000	Moist

Fig. E. Table program of predicted cambers (self weight of member only) for various concrete design strengths and curing methods.

varied from 3600 to 5000 psi.

To anticipate these rather variable conditions the predicting graphs as shown in Fig. E were developed. Surprisingly the actual cambers confirm the graphs as it was not possible to discern a member released at 3600 psi from one released at 5000 psi by observing the measured cambers.

An earlier camber study by Salmons confirms these observations.⁴⁰ Table B suggests how the various loss factors may interact to dampen the combined influence.

Fig. F summarizes the results of a study well outside the normal product

range. Accuracy for such conditions reinforces confidence in the recommendations made by the Committee for normal situations.

Table C summarizes a study of old projects ranging in age from 1 to over 10 years. While the previous examples were concerned with short-term behavior, these data confirm the long-term losses.

Fig. G (four sheets) contains a flow chart for a computer program which computes losses at the four suggested intervals. Since retrieving tabulated data is inefficient, all tables were replaced by empirical expressions. Another advantage of the expressions is that they offer con-

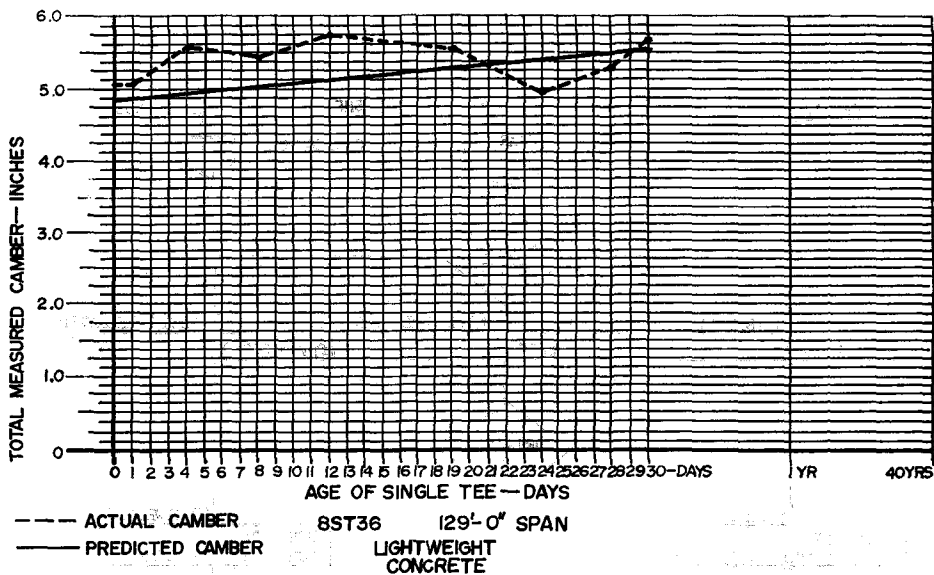


Fig. F. Comparison of actual camber versus predicted camber for a prestressed lightweight concrete single tee.

Table B. Net camber influence factors
(includes deflection due to gravity).

Recommendations Loss Variables		Camber		Deflection
		Creep	Shrinkage	
Concrete	Normal	?	-	+
	Lightweight	?	+	-
Cure	Moist	?	0	0
	Accelerated	?	0	0
Release f'_{ci}	High	?	+	+
	Low	?	-	-
Design f'_c	High	?	+	+
	Low	?	-	-

Legend

- + = increases net camber
- = decreases net camber
- ? = increase or decrease depending upon inelastic effects
- 0 = no change

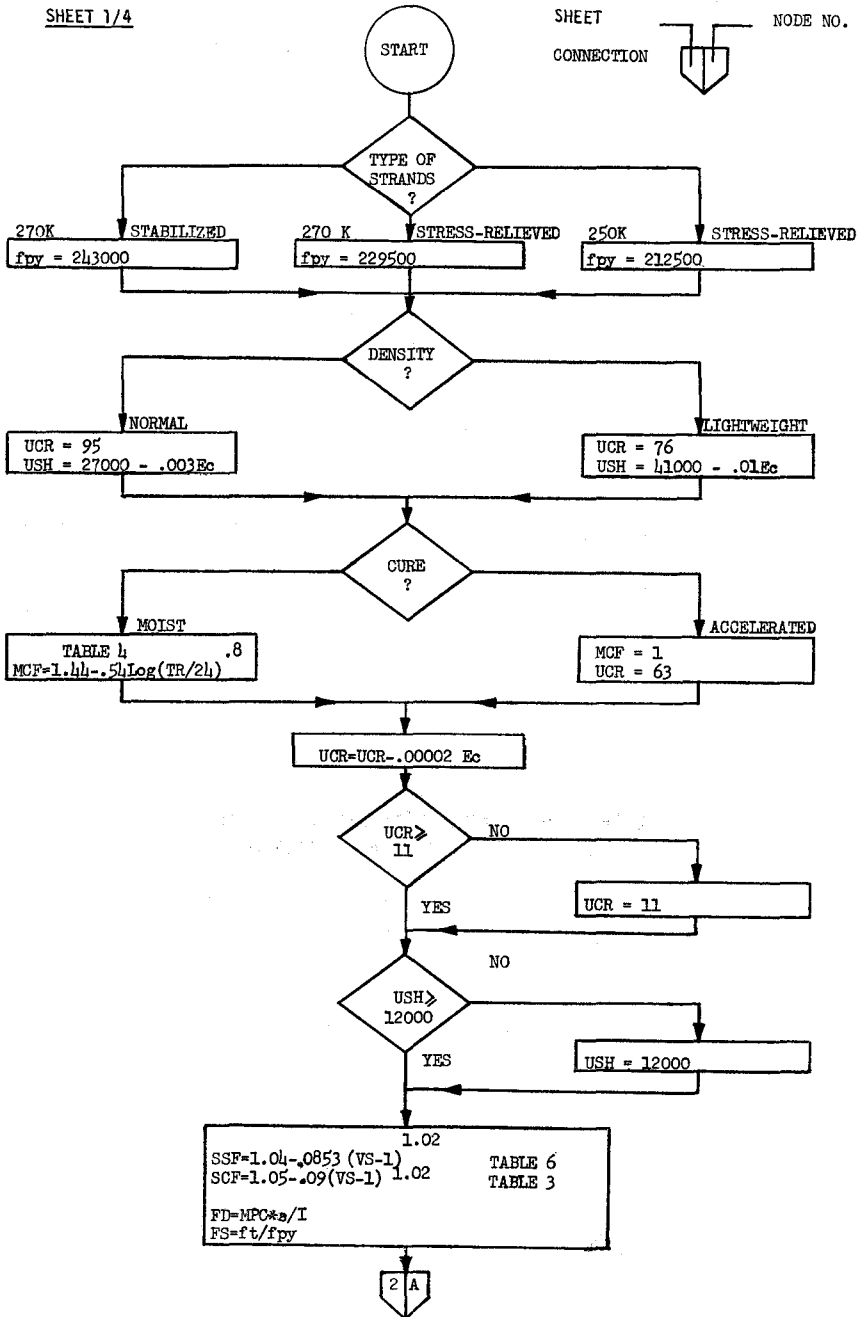


Fig. G. Flow chart for computing prestress losses.

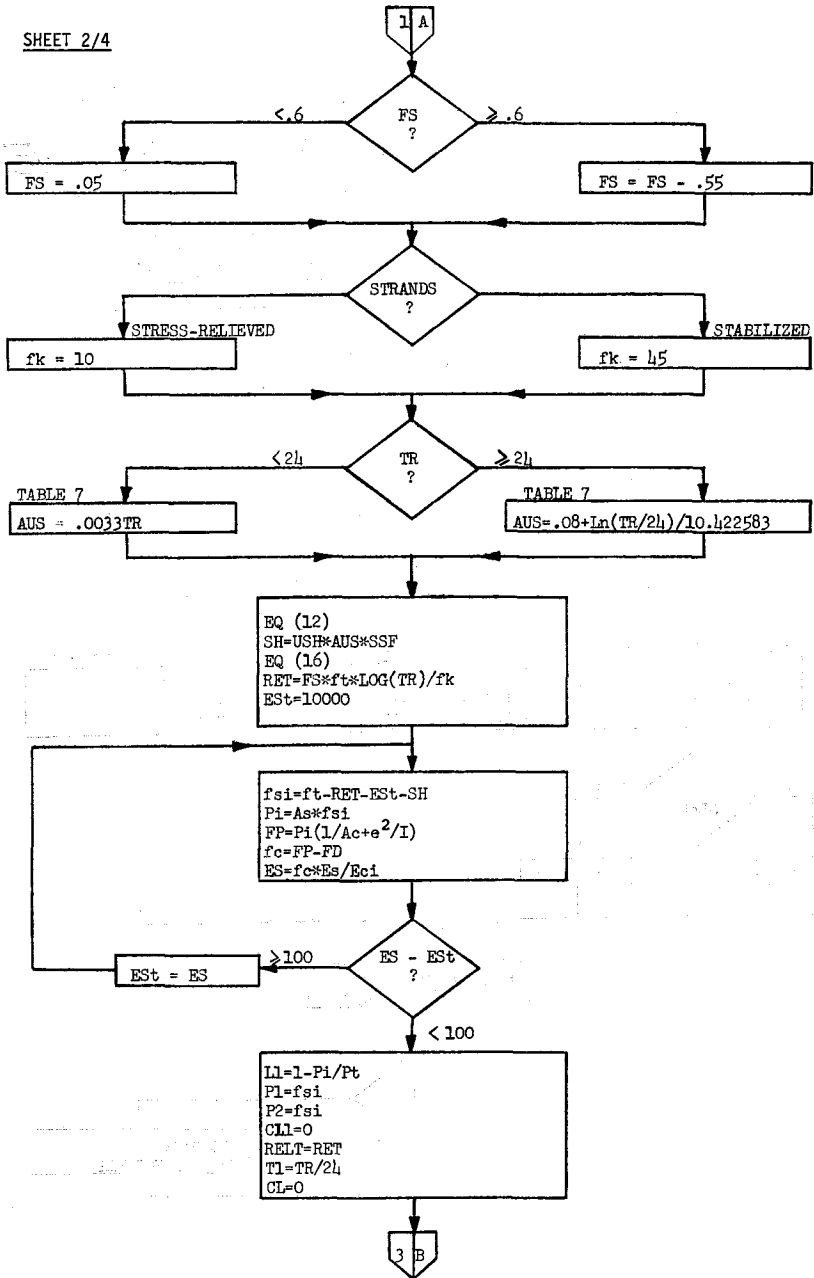


Fig. G. (cont.) Flow chart for computing prestress losses.

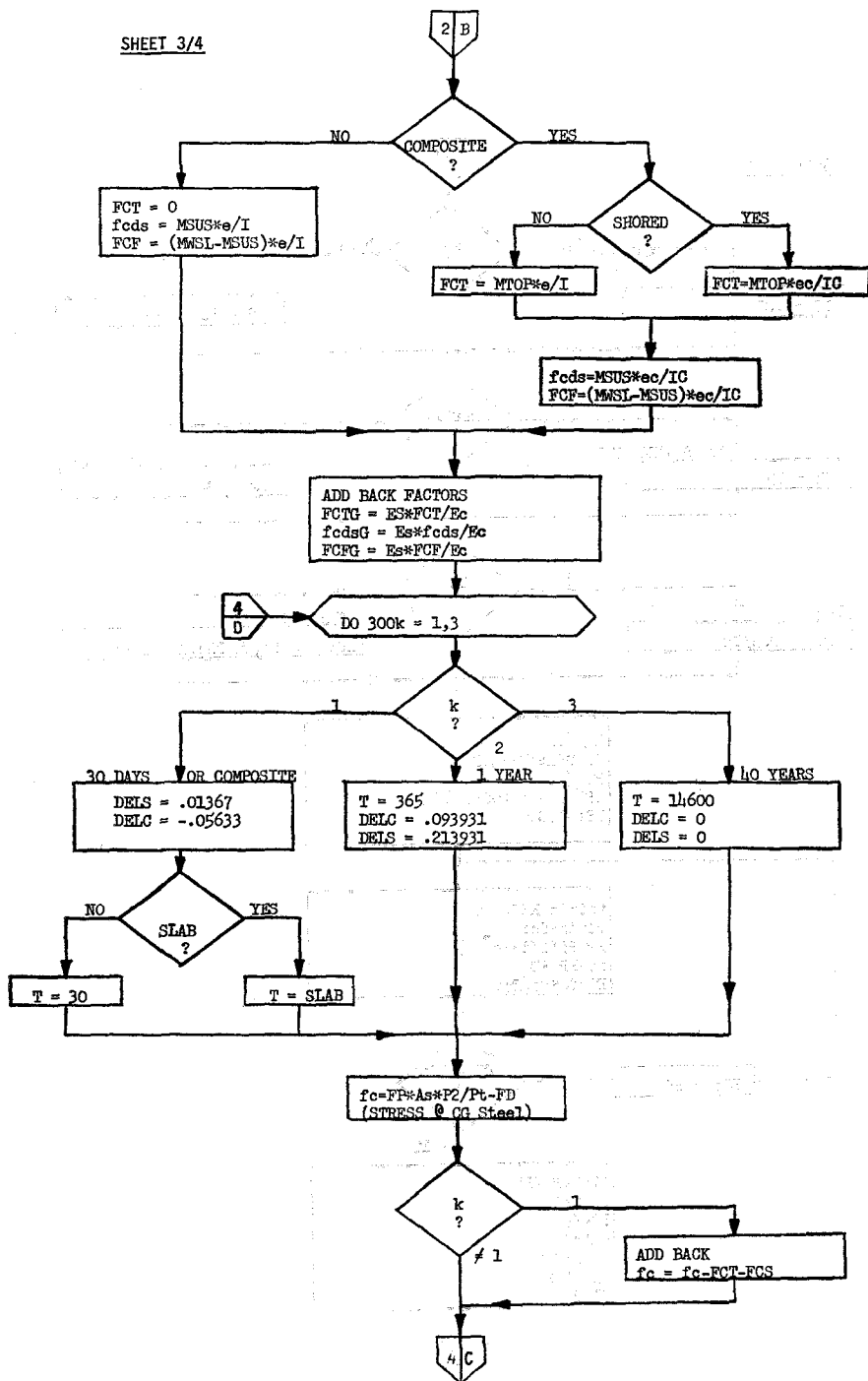


Fig. G. (cont.) Flow chart for computing prestress losses.

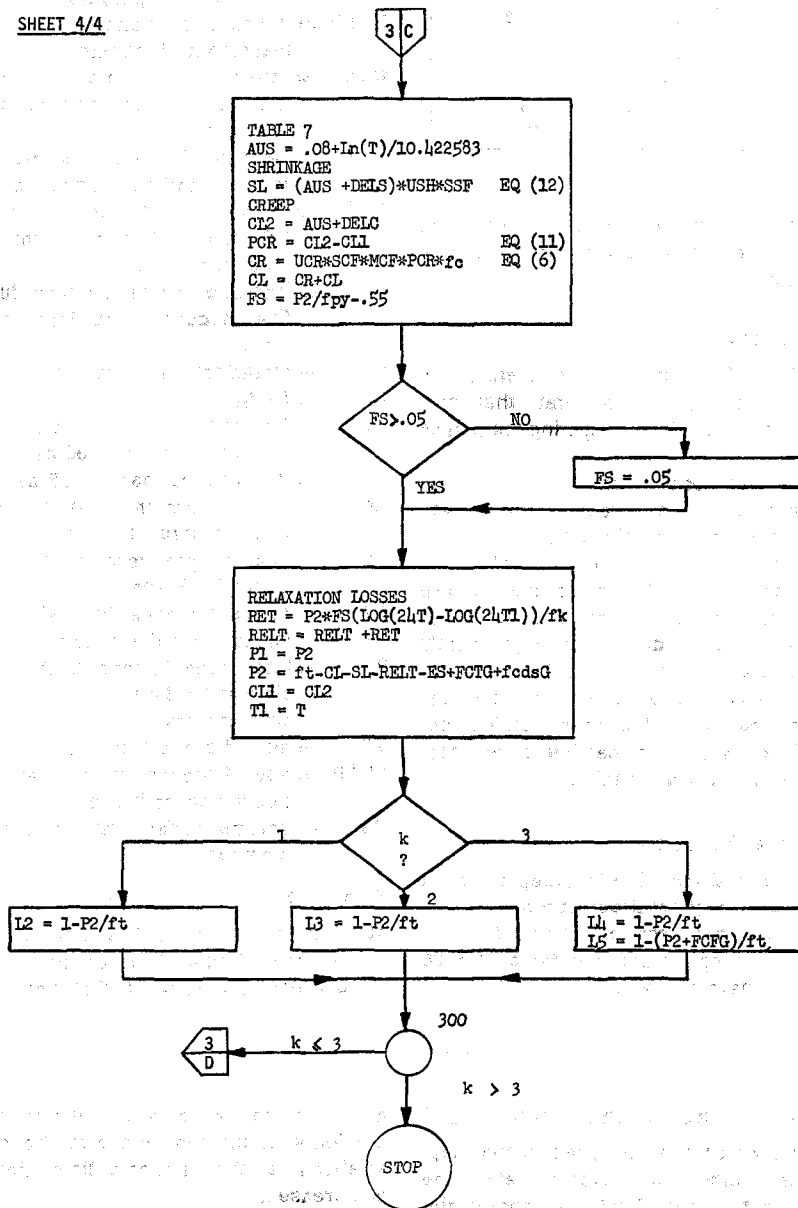


Fig. G. (cont.) Flow chart for computing prestress losses.

Table C. Predicted versus actual camber (old projects).

Product	Span, ft	Variance (Predicted-Actual) in.
8DT 24	53	0.08
6ST 36	98	0.06
6ST 36	107	1.00
8ST 36	100	0.11

venience in modifying the program to compute losses for a variable number of intervals.

Conclusions

1. The Committee's recommendations are significantly more accurate than previous methods for computing prestress losses.

2. Proper recognition of the time-dependent behavior will reduce or eliminate long-term deflection difficulties.

3. As related to camber and deflection, the total effect of varying the release strength, method of cure, and density is dampened and does not significantly change the net camber or deflection.

4. Unless the long-term camber/deflection behavior of a near identical design is known, use of camber growth factors can be very misleading.

Additional Notations

CL = total loss due to creep at time t
 CL1 = loss of prestress at time $t-1$
 CL2 = loss of prestress at time t
 DELC = creep adjustment factor for 30 days or 1 year

DELS = shrinkage adjustment factor for 30 days or 1 year

Est = estimated loss due to elastic shortening

FD = stress in concrete at Tendon CG due to self weight

FCT = stress in concrete at Tendon CG due to topping weight

FCTG = increase in strand stress due to placement of topping

fcds = stress in concrete at Tendon CG due to sustained superimposed load

fcdsG = increase in strand stress due to sustained superimposed load

FCF = stress in concrete at Tendon CG due to final superimposed load increment

FCFG = increase in strand stress due to final superimposed load increment.

fk = strand relaxation factor

FS = f_{st}/f_{py}

L1 = loss of prestress at release

L2 = loss of prestress at 30 days

L3 = loss of prestress at 365 days

L4 = loss of prestress at 40 years under sustained loads

L5 = loss of prestress at 40 years under full loads

P1 = stress in steel at time $t-1$

P2 = stress in steel at time t

Pt = total force in tendon at anchor

T1 = time at $t-1$ days

T2 = time at t -days

TR = age of precast at release, hours

SLAB = age of precast member at composite concreting, days

VS = volume surface ratio of precast member

Reference

- Salmons, John R., "Camber Study of 36-Inch Single Tee Section," PCI JOURNAL, V. 8, No. 5, October, 1963.

Maher K. Tadros and Amin Ghali*

Eq. (6), which is to be used in the step-by-step calculation, overestimates the loss due to creep. Firstly, it ignores the reduction in the stress f_e occurring during individual intervals. Thus, the elastic

and creep recoveries within the intervals are ignored; the resulting error becomes smaller when the number of time intervals is increased.

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Secondly, the stress f_c used for an intermediate interval is the algebraic sum of the effect of the initial prestress, dead load, and prestress loss occurring before the considered interval. Obviously, these stresses are not introduced at one age of concrete, and consequently they have different creep effects. Eq. (6) implies that all the stresses are applied at transfer and therefore have the same *UCR*, *MCF*, and *PCR*.

To reduce the above approximations requires an equation¹¹ more complicated than Eq. (6). The significance of the approximations is discussed here by considering the beam of Example 1 of the report (without concrete topping).

Using the same time intervals as in the report, the Committee method gives:

Stage	CR	SH	RET
2	4.17	5.05	5.11
3	3.90	5.29	2.13
4	2.20	1.68	1.77
Total	$10.27 + 12.02 + 9.01 = 31.30$ ksi		

The same example is solved with improved accuracy by accounting for the concrete aging and the recovery effect. The change in f_c is assumed to occur at the middle of each interval.¹¹ The modification factor for the effect of age at loading, *MCF*, used in the calculation of the results to follow is based on Reference 42. (Table 4 of the report is not applicable for the beam considered, being of steam-cured concrete.) With the same time intervals, the following is obtained:

Stage	CR	SH	RET
2	2.30	5.05	5.11
3	2.38	5.29	2.29
4	1.51	1.68	2.14
Total	$6.19 + 12.02 + 9.54 = 27.75$ ksi		

Increasing the number of intervals with the Committee method would give a more accurate value for the loss which is closer to 27.75 ksi.

In a recent paper¹³ two tables of "recovery" and "relaxation reduction" factors were derived using the improved step-by-step procedure described above. The use of the tables eliminates the necessity of performing the step-by-step computations. The accuracy achieved is the same as would be reached with a large number of intervals. The use of the tables for the above example gives a prestress loss of 26.81 ksi.

The writers agree with the Committee's statement in Section 1.4 concerning the composite action between precast and cast-in-place concrete parts. It is for this reason, and to separate this issue that in the above calculations the beam was considered without topping.

References

- Ghali, A., Tadros, M. K., and Dilger, W. H., "Accurate Evaluation of Time-Dependent Deflection of Prestressed Concrete Frames," American Concrete Institute, Publication SP-43, 1974, pp. 357-375.
- ACI Committee 209, Subcommittee 2, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures," American Concrete Institute, Publication SP-27, 1971, pp. 51-93.
- Tadros, M. K., Ghali, A., and Dilger, W. H., "Time-Dependent Prestress Loss and Deflection in Prestressed Concrete Members," PCI JOURNAL, V. 20, No. 3, May-June 1975, pp. 86-98.

Edwin B. Workman*

The Committee is to be commended for an excellent first cut at a comprehensive consideration of prestress losses. The report is a valuable contribution toward the understanding of the interaction of all

major loss sources. There appear to be a few minor flaws and a major omission, however.

It is apparent from a careful reading

*Chief Structural Engineer, Northeast Division, Atlas Prestressing Corp., Springfield, Virginia.

of the report that members prestressed by post-tensioning were included in the recommendations as an afterthought and hence are covered incompletely. The stated assumptions of compression levels, concrete strengths, and age at transfer do not represent most practical post-tensioned building members. These and other comments are not made specifically here, as they do not correct the overall shortcomings of the report.

The major flaw of the recommendations is the failure to adequately discuss the effects of prestress loss and the consequences of variation in prestress loss. No guidance is offered to warn a designer what type members may be sensitive to small variations in prestress, such as simply supported, shallow roof members which might fall under ponding rainwater, and what type of members have been safely designed considering nominal lump sum losses, as multi-span, continuous members which are insensitive to small changes in prestress. Yet the recommendations propose that four-stage loss calculations are necessary as routine design calculations.

Indeed, this writer is unaware of problems with underprestressed members except for cases where simple, single-span

members have been "stretched" to high span/depth ratios and used in roofs. Even here, prestress loss is only one factor of poor performance, along with lack of stiffness, and other causes.

To the contrary, most problems in prestressed structures have resulted from the exaggeration of creep and shrinkage effects by high initial prestress values. Hence, a "conservative" approach to prestress loss has increased structural distress. A calculated lump sum loss of 30 ksi for post-tensioned members represents a significant increase over the long accepted range of 24 to 27 ksi.

According to material distributed to PCI members regarding similar provisions for AASHTO use, these recommendations will add "only" about 4 percent more prestressing steel. The Committee apparently advocates this increase in cost in design time and materials without justification from actual building (poor) performance. Since the need for higher prestress loss values has not been demonstrated, even in a general way, these increased costs will have purchased nothing, and will in some cases actually amplify potential distress caused by shortening effects.

Committee Closure

The Committee appreciates the detailed comments offered by so many readers. These comments and responses should clear up many of the questions that arise in dealing with the complex problem of stress losses in prestressed concrete members.

Of particular interest are the comments by Messrs. Tadros-Ghali and Messrs. Gamble-Fadl which can reduce the values computed for stress loss due to creep and shrinkage of concrete. The designer should keep them in mind and apply them if they would make a significant difference in the structure under consideration. Overestimation of losses does not necessarily represent conservative design.

There is one factor that was overlooked

in both the General and Simplified Methods in the original report. The rate of relaxation of stress-relieved wire and strand increases rapidly with relatively small increases in temperature. At 140 F it is three times as great as at 68 F. All loss calculations in the report are based on 68 F.

If we assume that 270-grade strands in a pretensioned member have their temperature raised to 140 F from 6 hours after the concrete is cast until 18 hours after the concrete is cast, the net stress loss due to strand relaxation will be increased by about 3.5 ksi. Values for different temperatures and periods of time can be interpolated. The fact that the stress existing in the strand during the period of elevated temperature is about 13,000 psi less than at 68 F was included

in determining the additional net loss of 3.5 ksi.

Before presenting the specific responses to each of the readers, the Committee suggests that the Committee Statement in the original report be considered. Excerpts from the Statement which should be kept in mind include:

1. "A precise determination of stress losses in prestressed concrete members is a complicated problem."
2. "... the computed values for prestress loss are not necessarily exact, but the procedures here presented will provide more accurate results than by previous methods which gave no consideration to the actual stress levels in concrete and tendons."

Committee Response to Messrs. Gamble and Fadl

Designers should certainly give consideration to the statement in this discussion that losses due to shrinkage of exposed, unheated structures can be much less than the 12,000 psi minimum recommended in the report.

The 1975 Interim Specifications for Bridges published by AASHTO includes a new criterion for loss due to shrinkage of pretensioned bridge members which varies from 14,000 psi in parts of Southern California to 6500 psi in Illinois and 5000 psi in some east coast areas.

Use of the equations proposed for f_{si} and Δf_s will be more accurate than assuming $f_{si} = 0.90f_s$ as suggested in Section 3.2.5. However, the whole point of Chapter 3 is to keep the computations required to an absolute minimum.

The footnote to Section 3.2.5 suggests limits for heavily and lightly prestressed members and the last part of Design Example 2 shows how to check the accuracy of the assumed value of f_{si} and also shows the error in ES for an error of 10.0 ksi in the assumed value of f_{si} .

The Committee was aware of the fact that release strength rather than final ultimate often governs the concrete mix design and that the result is a final ultimate considerably in excess of specifications. Equations in Tables 8 and 9 were established by a large number of design examples processed through a computer

on the basis of the General Method. One of the variables checked in the examples was a 28-day strength of 6000 psi. As stated in Section 3.2.3:

"It was found that an increase in concrete strength at the time of prestressing or at 28 days made only a nominal difference in final loss and could be disregarded."

The discussion questions the relationship between the properties specified for strand, the properties of strand actually being fabricated, and the effect of any differences on relaxation computations. Eq. (16) is intended to represent the pure relaxation that will occur in a stress-relieved strand when the value of f_{py} substituted in the equation is $0.85f_{pu}$ and $f_{pu} = \text{GUTS}$ (Guaranteed Ultimate Tensile Strength).

The relaxation properties of stress-relieved strand vary with strand diameter as well as with strand fabricators. Fritz Engineering Laboratory Report No. 339.5 from Lehigh University is a 123-page report covering 40 relaxation tests on samples of 7/16 in. and 1/2 in. diameter 270-grade, stress-relieved strand furnished by three different strand fabricators.

Physical properties of the strands tested were in the following ranges:

7/16-in. diameter strand

Load at 1 percent elongation =
28,500 to 30,000 lb

(Specification minimum = 26,350 lb)
Actual ultimate = 32,820 to 35,800 lb
(Specification minimum 31,000 lb)

1/2-in. diameter strand

Load at 1 percent elongation =
37,500 to 38,500 lb

(Specification minimum = 35,100 lb)
Actual ultimate = 42,500 to 42,710 lb
(Specification minimum = 41,300 lb)

Tests were conducted at initial stress levels of 50, 65, and 80 percent of GUTS. Duration of tests was approximately 15 months. Conclusions included the following statements:

1. Strands from Manufacturer C have the lowest potential for relaxation loss, while strands from Manufacturer B suffer the highest loss.

2. In general, the 1/2-in. diameter strands undergo higher losses than the 7/16-in. diameter strands.

3. The relaxation loss depends strongly on the initial stress in the strand. There is an indication that losses are not negligible even for initial stresses below 0.50 f_{pu} .

In the body of the report a discussion of behavior in the stress range of 0.50 f_{pu} to 0.70 f_{pu} goes on to say "Above this range, the loss increases rapidly with increasing initial stress and the behavior deviates sharply . . ."

Table D gives a comparison of actual test results from the Lehigh University report and corresponding values computed by Eq. (16) using $f_{py} = 0.85$ GUTS. All values are for an initial tension of 0.65 f_{pu} . Test values for 1.0, 20, 50, and 100 days are from Fig. 20 of the Lehigh report. Actual results beyond 100 days were not shown.

The tonnage of 1/2-in. diameter strand used in the United States today is several times that of all the smaller sizes put together. The maximum amount by which Eq. (16) exceeds the average test values in Table D for 1/2-in. diameter strand is

0.60 percent at 20 days.

The maximum amount by which the average test values for 1/2-in. diameter strand exceeds Eq. (16) is 0.10 percent at 100 days. All values computed by Eq. (16) are within the spread of the test results.

Eq. (17) represents the actual properties of the low relaxation strand being marketed in the United States. Test data are available from the manufacturers.

Committee Response to Mr. Gregg

Mr. Gregg's suggestions for combining values such as using one curve for AUC and AUS are practical and certainly within the accuracy of currently available data. His equations for SCF and SSF are also of adequate accuracy. These simplifications should be helpful to designers setting up computer programs to compute losses by the General Method.

Committee Response to Mr. Jenny

Mr. Jenny's discussion of "add-back" deals comprehensively with one of the

Table D. Comparison of test results from Lehigh University with Eq. (16).

Elapsed Time, days	Test Results*						Equation**
	7/16 in. Diameter			1/2 in. Diameter			
	Min. percent	Max. percent	Avg. percent	Min. percent	Max. percent	Avg. percent	
1.0	2.0	2.2	2.1	2.4	3.0	2.7	2.96
20	4.0	5.0	4.5	4.4	5.9	5.15	5.75
50	5.0	5.9	5.45	6.2	7.1	6.65	6.60
100	6.0	8.0	7.0	6.4	8.3	7.35	7.25

Notes: Pure relaxation of stress-relieved strand.

Initial tension = 65 percent guaranteed ultimate tensile strength.
Losses shown are in percent of initial tension.

* Actual results of 40 relaxation tests on stress-relieved strand from Lehigh University Lab Report 339.5.

** Equation from AASHTO - 1975, p. 50 and from PCI JOURNAL, July-August, 1975, p. 51.

$$\Delta R = \frac{\log t}{10} \left[\frac{f_{si}}{f_y} - 0.55 \right]$$

ΔR = pure relaxation as a percent of initial tension

t = elapsed time in hours

f_{si} = initial stress

f_y = guaranteed minimum yield stress at 1.0 percent elongation

controversial questions that had to be resolved by the Committee.

"Add-back" can be defined as "Increase in tensile stress in the tendons resulting from an increase in flexural stress in the prestressed concrete member." Since the stress loss due to tendon relaxation is a function of the stress existing in the tendon, anything that causes a change in tendon stress could have an effect on the net stress loss.

The question was how should add-back be treated in the determination of stress losses? As shown in Design Example 1, Stage 2, the strand stress is increased by 5048 psi due to the weight of the topping. In the "Summary of Steel Stresses" the total loss is shown as 48.57 ksi but this is then reduced by the 5.05 ksi due to the topping before it is subtracted from the initial stress of 189.0 ksi to get the new effective tendon stress under dead load of 145.48 ksi. Therefore, the add-back due to *permanent loads* has been included in computing the net effective tendon stress under dead load.

In the Simplified Method the equations in Table 8 give total losses without any add-back. The equations in Table 9 give net effective tendon stress under dead load after all losses and include an add-back for increase in tendon stress due to permanent loads applied to the member after it has been prestressed.

If the designer wishes he may compute flexural stresses on the basis of the transformed section produced by bonded tendons but only for those loadings where add-back is not used. Thus, transformed section may be used in connection with all transient live loads and with dead loads where stress losses are based on total prestress loss. Gross concrete section should be used for computation of dead load stresses where the net effective prestress is used as with the equations from Table 9.

Committee Response to Mr. Mallett

Mr. Mallett has accepted the report for what it is intended, namely, a method that is significantly more accurate than the old method.

He then ties it in with the development

of long-term camber/deflection behavior and offers data that will be of use to designers concerned with that problem.

Be sure to heed the warning at the end of his sixth paragraph. Camber and deflection characteristics for presumably identical members from different plants are not necessarily the same. They may also differ for long span and short span members from the same plant. Past experience with similar members should be considered whenever possible.

Committee Response to Messrs. Tadros and Ghali

This discussion is correct in stating that the stress loss due to creep of concrete is overestimated by basing the creep loss during an interval on the compressive stress around the tendons at the beginning of the interval. It is also correct in stating that the degree of accuracy will increase as the number of intervals used is increased.

The accuracy of a calculation using only four intervals could be improved by estimating the change in concrete stress that will take place during the interval and using the average value during the interval when computing creep loss for the interval.

The second paragraph of the discussion objects to the idea that "Eq. (6) implies that all the stresses are applied at transfer and therefore have the same *UCR*, *MCF*, and *PCR*."

The discussers stated correctly that, at any one time, the concrete stress f_c is a combination of several effects each originating at a different time. In the General Method, the total creep strain is estimated based on the principle of creep rate (as opposed to superposition), and the effect of differential initial time of the several stress components is ignored.

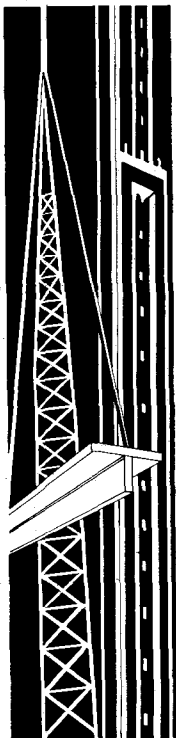
The resulting error slightly underestimates the creep effect of all stresses except the initial prestress. However, the error is small and tolerable, as was demonstrated by the discussers' examples.

Incidentally, Eq. (6) of the report does not imply, as the discussers suggest, that all stresses are applied at transfer, only that all stresses are associated with the same strain rate.

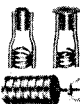
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(cont. from p. 125)

Committee Response to Mr. Workman

Mr. Workman states that "The major flaw of the recommendations is failure to adequately discuss the effects of prestress loss and the consequences of variation in prestress loss."

The Committee Statement calls attention to the fact that "An error in computing losses can affect service conditions such as camber, deflection and cracking."

The Committee was well aware of the fact that an accurate determination of final tendon stress is much more important in some members than in others. There were other items concerning design and fabrication that also might have been discussed in the report. However, the Committee's mission was solely to provide a procedure for determining losses. The significance of the results should be evaluated by the designer for the particular structure on which he is working.

Section 2.3 of the Commentary reflects the basic philosophy of the report when it says "It is not desirable to be conservative . . ." The object of the procedure is to determine, as accurately as possible, the actual stress in the tendons for the conditions applicable to the specific structure.

For members with an average amount of net compressive stress under dead load, the combination of the recently established higher allowable tensile stresses in concrete with the higher stress losses determined by the report does not increase the amount of tendons required. Heavily prestressed members have more loss due to creep of concrete and do require more tendons. Lightly prestressed members frequently require less tendons than by the old method.

Neither the Recommendations nor the Commentary mention a lump sum loss of 30 ksi for post-tensioned members.

If an engineer is consistently designing members of a particular type, such as with low prestress, he could use the General Method for several designs with different intensities of prestress and plot a curve from which losses could be read without further computation.

Calendar of Coming Events

May 6-8, 1976—Precast Concrete Drafting Seminar, Prestressed Concrete Institute, Bismarck Hotel, Chicago, Illinois.

June 5, 1976—T. Y. Lin Symposium on Prestressed Concrete, Physical Sciences Lecture Hall, University of California at Berkeley. (For further details see p. 106 of current issue of PCI JOURNAL.)

June 21-25, 1976—Recent Advances in Earthquake-Resistant Design of Structures, University of California at Berkeley.

August 19-21, 1976—International Symposium on Earthquake Engineering, The University of Missouri-Rolla, St. Louis, Missouri. (Contact Dr. Franklin Y. Cheng at University of Missouri-Rolla, Rolla, Missouri 65401, Tel. 314-341-4469).

August 28-September 3, 1976—FIP Symposia on Bridges and Buildings, Federation Internationale de la Précontrainte, Sydney Hilton Hotel, Sydney, Australia. (Contact: FIP Secretariat, Terminal House, Grosvenor Gardens, London, England SW1WoAU.)

October 17-21, 1976—PCI Annual Convention, Prestressed Concrete Institute, Americana Hotel, Bal Harbour, Florida.

October 24-29, 1976—Fall Convention, American Concrete Institute,

Mexico, D. F., Mexico (Cosponsored by Instituto Mexicano del Cemento y del Concreto, A. C.)

November 17-19, 1976—Building and Construction Exposition and Conference, McCormick Place, Chicago, Illinois. (Contact: The Producers' Council, Inc., 1717 Massachusetts Avenue, N.W. Washington, D.C. 20036.)

March 27-April 1, 1977—ACI Annual Convention, American Concrete Institute, Town and Country Hotel, San Diego, California.

October 2-6, 1977—PCI Annual Convention, Prestressed Concrete Institute, Royal York, Toronto, Ontario.

April 30-May 5, 1978—FIP VIII/Congress, Fédération Internationale de la Précontrainte, London, England.

September 17-21, 1978—PCI Annual Convention, Prestressed Concrete Institute, Hyatt Regency, New Orleans, Louisiana.

October 14-18, 1979—PCI Annual Convention, Prestressed Concrete Institute, Hyatt Regency, Dallas, Texas.

October 12-16, 1980—PCI Annual Convention, Prestressed Concrete Institute, Washington Hilton Inn, Washington, D.C.

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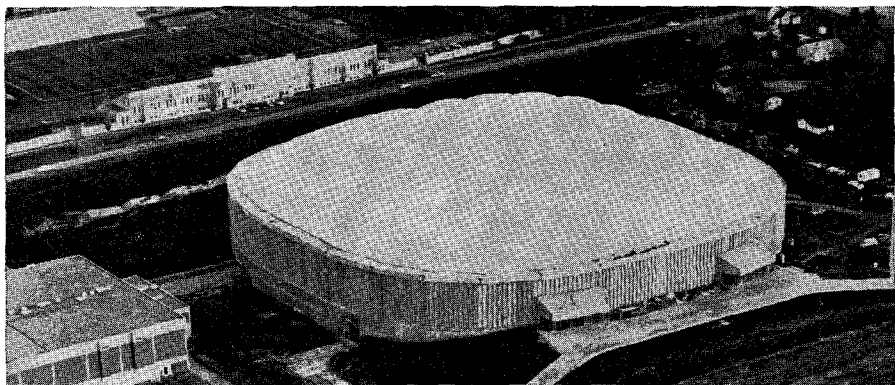
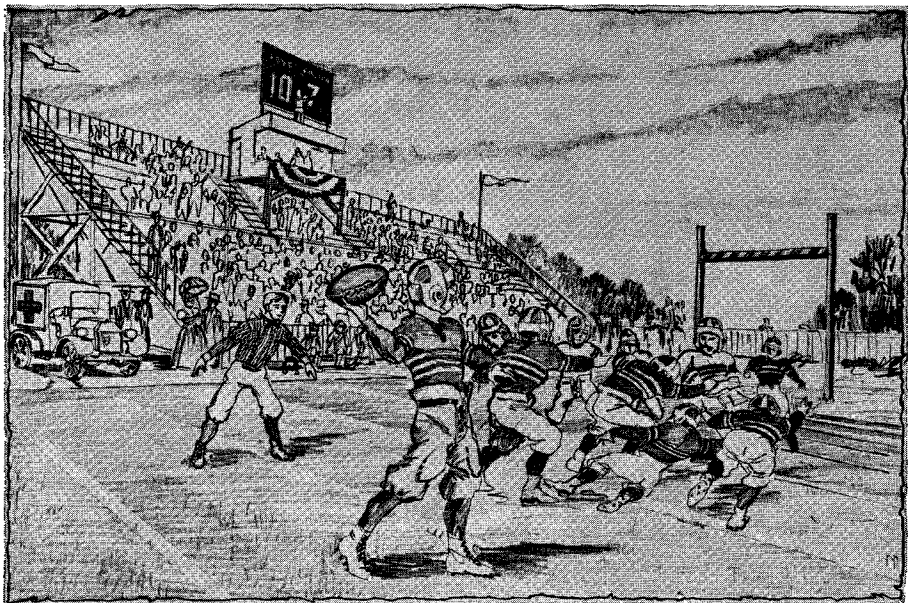
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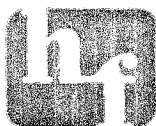
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