TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-84/32+248-1		

4. Title and Subtitle

EXPLORATORY STUDY OF SHEAR STRENGTH OF JOINTS FOR PRECAST SEGMENTAL BRIDGES

5. Report Date

September 1983

6. Performing Organization Code

7. Author(s)

K. Koseki and J. E. Breen

8. Performing Organization Report No.

Research Report 248-1

9. Performing Organization Name and Address

Center for Transportation Research
The University of Texas at Austin
Austin, Texas 78712-1075

10. Work Unit No.

11. Contract or Grant No.

Research Study 3-5-80-248

12. Sponsoring Agency Name and Address

Texas State Department of Highways and Public Transportation; Transportation Planning Division
P. O. Box 5051
Austin, Texas 78763

13. Type of Report and Period Covered

Interim

14. Sponsoring Agency Code

15. Supplementary Notes

Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration. Research Study Title: "Reevaluation of AASHTO Shear and Torsion Provisions for Reinforced and Prestressed Concrete"

16. Abstract

The joints between the precast segments are of critical importance in segmental bridge construction. They are critical to the development of structural capacity by ensuring the transfer of shear across the joints and often play a key role in ensuring durability by protecting the tendons against corrosion. However, construction of the joints must be simple and economical. A number of types of joint configurations have been used in various precast segmental bridges in the United States, although relatively little information is available on the behavior and design of such joints. This study reports on a modest scope experimental investigation to determine the relative shear transfer strength across different types of joints typically used between adjacent segments of precast segmental bridges. The types of joints considered included no keys, single large keys, and multiple lug keys. Both dry and epoxy joints were tested. The test results indicated substantial differences in the strength at a given slip in the various types of dry joints, but indicated that all types of joints with epoxy essentially developed the full strength of a monolithically cast joint.

17. Key Words	18. Distribution Statement
joints, shear strength, segmental bridges, precast, no keys, single large keys, multiple lug keys, dry, epoxy	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	106	

Form DOT F 1700.7 (8-69)

EXPLORATORY STUDY OF SHEAR STRENGTH OF JOINTS FOR PRECAST SEGMENTAL BRIDGES

by

K. Koseki and J. E. Breen

Research Report No. 248-1

Research Project No. 3-5-80-248

Reevaluation of AASHTO Shear and Torsion Provisions for Reinforced and Prestressed Concrete

Conducted for

Texas

State Department of Highways and Public Transportation

In Cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

September 1983

P R E F A C E

This report presents the results of an exploratory study which considered the behavior and criteria for design of shear keys for segmental prestressed concrete box girder bridges. This study forms a part of a larger study which is reevaluating the basic AASHTO shear and torsion provisions for reinforced and prestressed concrete and stems directly from review comments wherein FHWA asked the researchers to consider the criteria for design of such shear keys in the overall study. The objective of the program reported herein was to review existing data and to conduct a limited scope experimental program to determine relative shear transfer strength across different types of joints between adjacent segments typical of precast segmental bridges. The types of joints considered included single large key, multiple lug keys, and joints with no keys. Both dry and epoxy joints were studied.

The work was sponsored by the Texas State Department of Highways and Public Transportation and the Federal Highway Administration and administered by the Center for Transportation Research at The University of Texas at Austin. Close liaison with the State Department of Highways and Public Transportation has been maintained through Mr. Warren A. Grasso and Mr. Dean W. Van Landuyt who served as contact representatives during the project and with Mr. T. E. Strock of the Federal Highway Administration.

The project was conducted in the Phil M. Ferguson Structural Engineering Laboratory located at the Balcones Research Center of The University of Texas at Austin. The authors would like to acknowledge the assistance of Figg and Muller Incorporated of Tallahassee, Florida, who provided detailed information about the multiple key joint configuration used in the study and Kajima Corporation of Tokyo, Japan, who provided financial support for Mr. Koseki throughout his study. The authors were particularly indebted to Mr. Gorham W. Hinckley, Laboratory Technician at the Ferguson Laboratory, who greatly helped in carrying out the laboratory work involved.

S U M M A R Y

The joints between the precast segments are of critical importance in segmental bridge construction. They are critical in the development of structural capacity by ensuring the transfer of shear across the joints and often play a key role in ensuring durability by protecting the tendons against corrosion. However, construction of the joints must be simple and economical. A number of types of joint configurations have been used in various precast segmental bridges in the United States, although relatively little information is available on the behavior and design of such joints. This study reports on a modest scope experimental investigation to determine the relative shear transfer strength across different types of joints typically used between adjacent segments of precast segmental bridges. The types of joints considered included no keys, single large keys, and multiple lug keys. Both dry and epoxy joints were tested. The test results indicated substantial differences in the strength at a given slip in the various types of the dry joints, but indicated that all types of joints with epoxy essentially developed the full strength of a monolithically cast joint.

IMPLEMENTATION

The results of this study will assist the designer to assess the merits of various types of joints proposed for this popular type of construction. Current design specifications do not address this problem and these results, while representing only a limited exploratory study, do indicate important trends and allow the bridge designer to better understand the trade-offs that are being made in the choice of joint type.

CONTENTS

Chapter Page

T A B L E S

FIGURES

CHAPTER 1

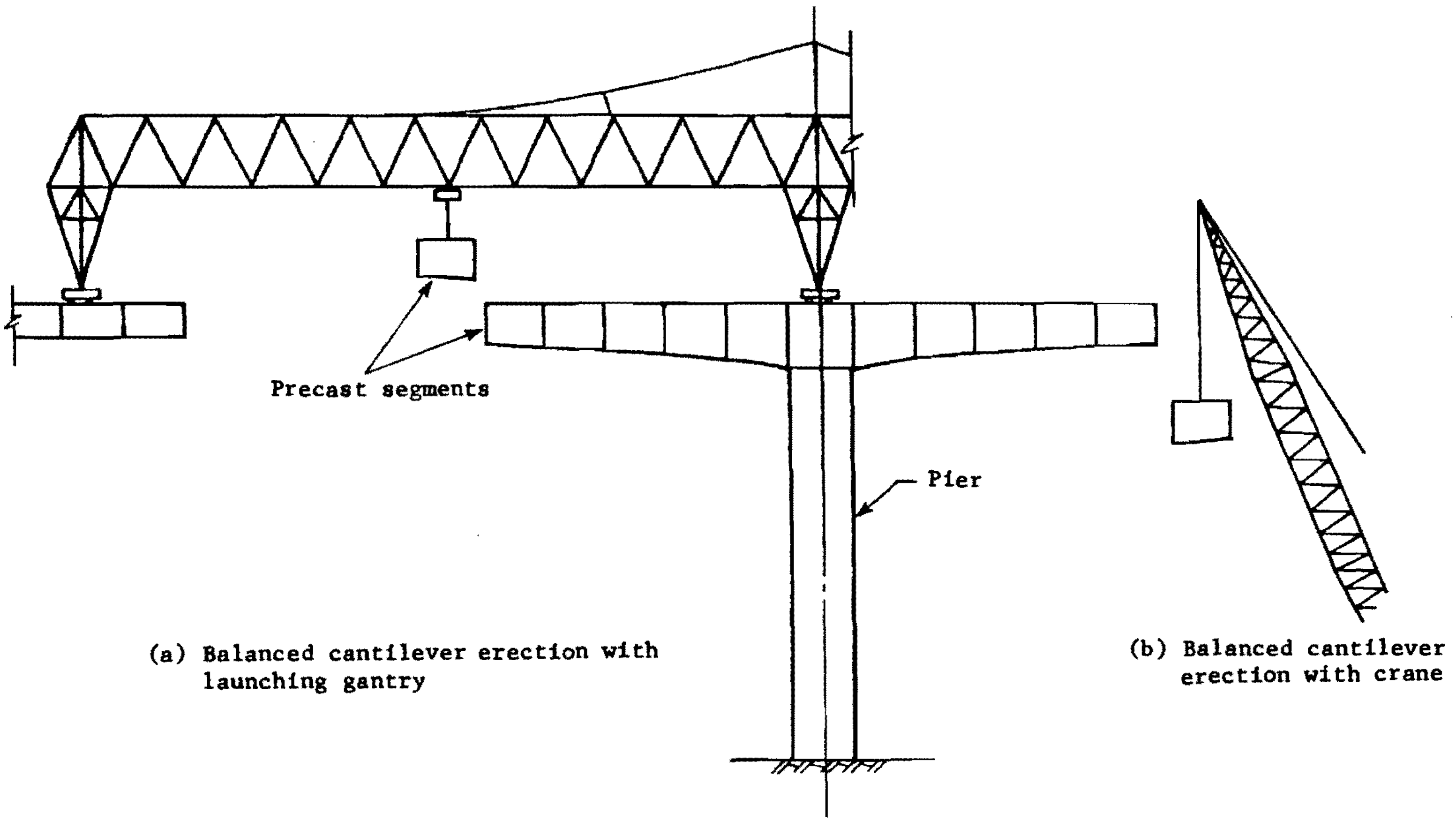
INTRODUCTION

1.1 Precast Segmental Bridge Construction

In precast segmental bridge construction, the structure is constructed by post-tensioning together precast segments which are usually manufactured as short longitudinal sections of the box girder cross section. Balanced cantilever erection (see Fig. 1.1) was the early predominant method of constructing segmental bridges. In a number of recent applications, the span-by-span method with segments assembled on a falsework truss has seen wide use.

The technology of precast segmental construction was an extension of cast-in-place segmental prestressed construction which was developed by Ulrich Finsterwalder and the firm of Dyckerhoff & Widmann A.G. (Dywidag) in West Germany in the 1950's [1,2]. The first major application of precast segmental construction was in the Choisy-le-Roi Bridge in 1962 [1,2]. The structure was designed by Jean Muller and the firm of Entreprises Campenon Bernard in France. Thereafter, the techniques of precasting segments and assembling them in the structure have been continually refined.

Precast segmental construction was introduced to the United States in the early 1970s. The JFK Memorial Causeway in Corpus Christi, Texas, was the first application of the method and was completed in 1973 [1,2]. Since 1975, this technique of constructing bridges has gained rapid acceptance, and there are presently over 80 such bridges either completed, under construction, or in design in North America [3]. During the initial development of segmental construction the bridges were constructed by the balanced cantilever method. Currently, such techniques as span-by-span construction, incremental launching, and progressive placing are also being utilized. The Long Key Bridge in Florida, the Wabash River Bridge in Indiana, and the Linn Cove project in North Carolina are examples of each of these procedures, respectively.



(a) Balanced cantilever erection with launching gantry

(b) Balanced cantilever erection with crane

Fig. 1.1 Typical precast segmental bridge construction

A large number of precast segmental bridges use an epoxy resin jointing material between precast segments. The thickness of the epoxy joint is on the order of 1/32 in. The use of an epoxy joint requires a perfect fit between the ends of adjacent segments. This is achieved by casting each segment against the end face of the preceding one (match-casting), and then erecting the segments in the same order in which they were cast.

While numerous examples of successful projects with such joints exist, there are also a number of possible disadvantages in precast segmental construction:

--Necessity for a high degree of geometry control during fabrication and erection of segments.

--Potential joint weakness due to lack of mild steel reinforcement across the joint.

--Temperature and weather limitations regarding mixing and placing epoxy jointing material.

--Frequent loading and unloading of segments, with the risk of damage.

The large number of successful projects in Europe, North America, and other parts of the world suggest that these obstacles will not curb the rapid growth in the use of precast segmental bridge construction. Epoxy joints, grouted tendons, and shear keys have reduced dependence on the bonded mild steel joint reinforcement, while the versatility of the match-casting procedure in numerous major projects involving complex horizontal and vertical alignment has shown that the precast procedures can deal with geometrical problems. A number of recent projects have been built with multiple key dry joints to eliminate epoxy coatings and their attendant problems.

1.2 Objective and Scope

1.2.1 Shear Keys and Epoxy Bonding Agent. The joints between the precast segments are of critical importance in segmental bridge construction. They must have high strength to transfer shear. If tendons pass through the joints, then they must have assured durability in order to protect the tendons against corrosion. In addition, construction of the joints must be reasonably easy and not overly sensitive to atmospheric conditions.

In match-cast segments, single or multiple keys are almost always used in the webs, with epoxy resins often used to coat the contact surfaces between adjacent segments.

The web keys serve two functions. The first is to align the segments during erection. The second is to transfer the shear force between segments during that period while the epoxy applied to the joint is still plastic and acts only as a lubricant. At that time, the web shear keys alone must be relied upon to transfer the shear force across the joint, since the fluid epoxy bonding agent minimizes the coefficient of friction between the segments. Examples of single and multiple keys are shown in Fig. 1.2. A typical multiple key joint has a series of small interlocking keys over the entire web height. This arrangement was developed to relieve the cured epoxy bonding agent of any structural function [4,5]. Use of internal stiffeners on the webs provides an anchor zone for the permanent prestressing tendons, thus moving them from the face of the web and permitting use of the multiple key design.

In some bridges, however, shear keys have not been used. In the Pasco-Kennewick Intercity Bridge, the match-cast joint contains no shear keys [6,7]. To facilitate alignment, 2-in. diameter steel pintles were used in the top slab at each joint. The segments were bonded together with epoxy. The shear force acting on the joint is always smaller than 5% of the longitudinal force because of the multi-cable stay system.

The major function of epoxies used in segmental joints is fourfold:

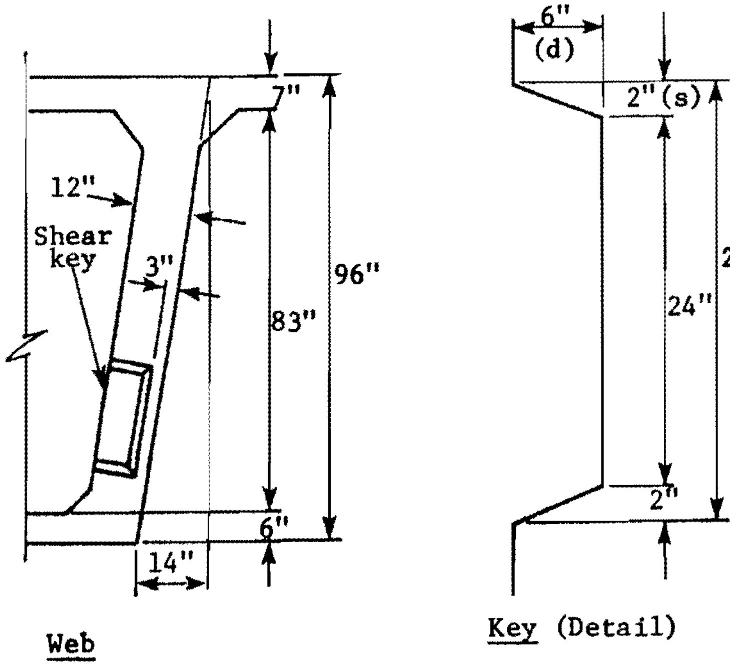
<u>In the liquid state</u>

- To act as a lubricant which facilitates jointing.
- To even out minor irregularities between the mating surfaces and to perfectly match the adjoining segments.

<u>In the cured state</u>

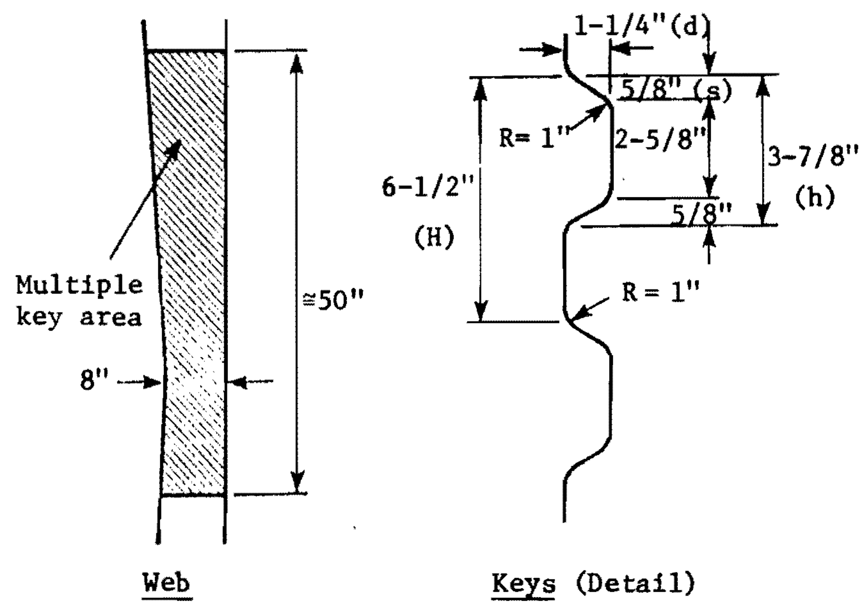
- To provide water tightness and durability at the joint and to protect the post-tensioning tendons running through the joint from corrosion.
- To transfer shear forces and to contribute to the structural rigidity.

a) JFK Memorial Causeway, Corpus Christi (Single key)



Key Area / Web Area	.25
d/s	3/1
d/h	.21

b) Long Key Bridge (Multiple keys, dry joint)



Number of Keys	9
h/H	.60
d/s	2/1
d/h	.32

Fig. 1.2 Examples of shear keys

Although in the early development of segmental construction, the main purpose of the epoxy bonding agent was to transfer shear stresses between adjacent segments, proponents of the use of multiple shear keys claim that this is no longer necessary. In fact, some recent bridges have not used the epoxy bonding materials. For example, in the Long Key Bridge in Florida, the use of epoxy was omitted [8]. Dry joints were used with span-by-span erection, internal location of the tendons, non-freeze-thaw climate, and use of multiple shear keys.

Epoxy joints with a single large key in each web have been widely used on early projects in the U.S. Most applications have been successful. However, a serious joint failure occurred on a bridge across the Kishwaukee River in Illinois [9]. The epoxy failed to harden, lubricated the joint, and caused cracks and spalling in a web where a singly keyed joint was used. The failure was attributed to improper blending and insufficient mixing of the epoxy resin and hardener. This indicates that improper use or choice of the epoxy can be critical with respect to shear strength of the joint.

1.2.2 Objective. Due to the rapidly growing number of segmentally constructed bridges, there is a need to better understand the characteristics of the different shear key configurations and the role of the epoxy bonding agent in shear transfer. The overall success of precast segmental construction will depend heavily on the behavior of the joints.

The objective of this exploratory study was to determine the relative shear transfer strength across different types of joints commonly used in precast segmental bridges. Types of joints considered included single large key, multiple lug-keys and no-key joints. Both dry and epoxied joints were studied.

Currently, "shear friction" concepts are often used in the design of joints. Thus, the dry joint with no-keys was included to allow results to be related to shear friction theory. All specimens were compared to the behavior of monolithically cast specimens to indicate relative efficiencies. The test series was envisioned as an exploratory series and did not investigate either the wide range of formulations available for epoxies or consider the wide range of loading conditions possible in service.

1.2.3 Shear Test. To accomplish the above-stated objective, seven jointing conditions for the test specimens were determined, as shown in Table 1.1.

Since the shear force acting on a box girder section is carried primarily by the webs, simple rectangular web model sections were used in the tests. Dimensions of the small-scale model specimens are the same as those of the 1/4 scale model used by Stone for the study of post-tensioned anchorage zone tensile stresses [10]. Figure 1.3 shows the relationship between typical box girder sections, a prototype web section, and the model web section used.

In order to obtain the relative shear transfer strength across the joints, the specimens were subjected to a predominantly shear test using the loading scheme shown in Fig. 1.4. This corresponds to a low a/d ratio. Since distributed load applications provide smaller maximum moment and less bearing stresses than concentrated ones, while giving the same amount of shear force at the joint, the load was applied in that manner as shown in Fig. 1.4(b).

Since time and resource requirements restricted the magnitude of this exploratory study, only one test specimen was made for each jointing condition. Figure 1.5 illustrates the fabrication sequence of the model precast segments and the test specimens made out of those model segments. Fabrication methods for the model segments and the details of the test will be described in the following two chapters. The test results will be discussed in Chapter 4.

1.3 Previous Related Studies

Several research papers which deal directly or indirectly with the subject studied herein have been published to date. Some of them are reviewed below, and the results obtained from those studies will be referred to later in Chapter 4.

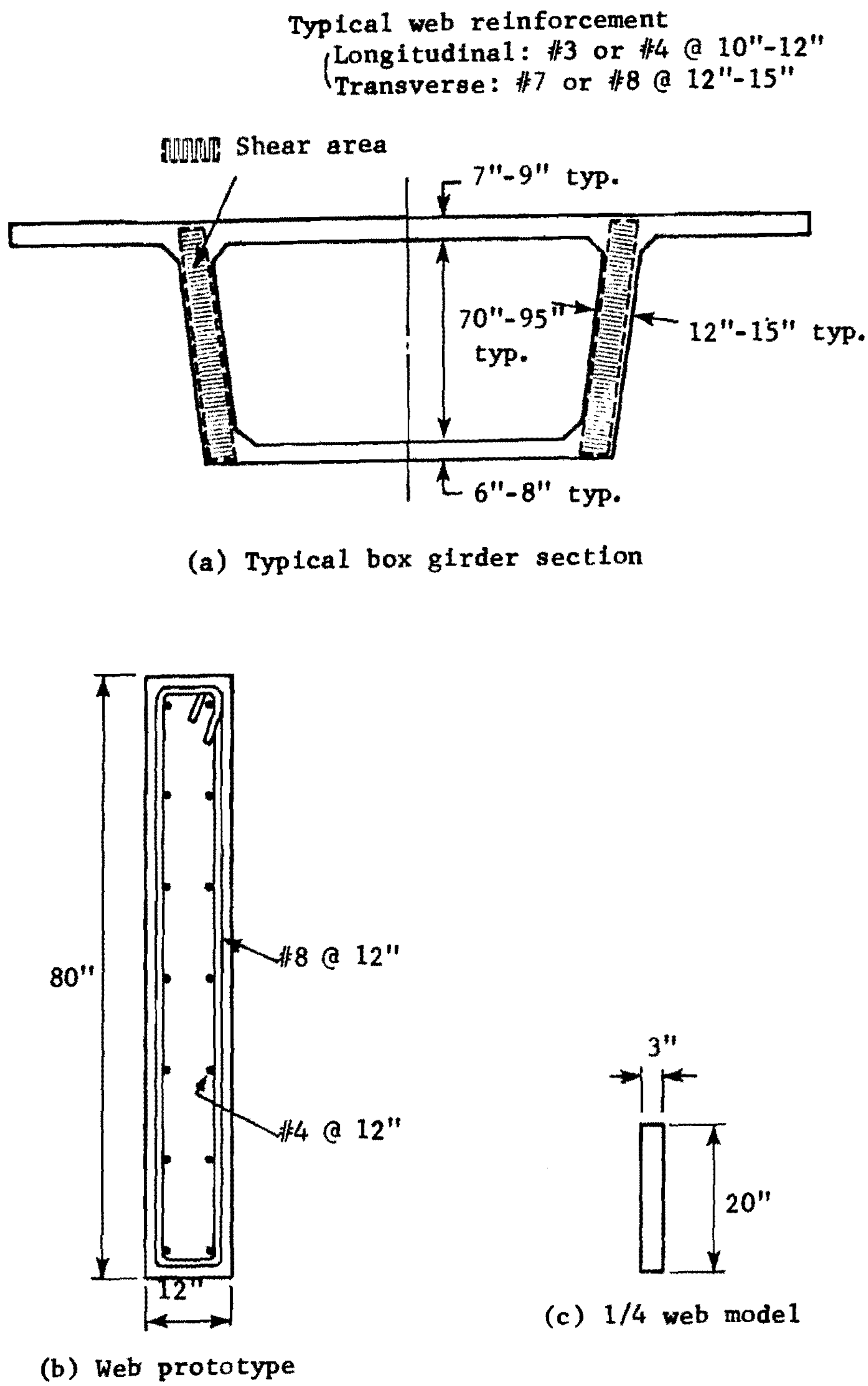
1.3.1 Shear Friction. Shear friction theory applications for precast connections are based on the work done by Birkeland and Birkeland [11] and Mast [12] at ABAM Engineers, Inc., and Concrete

TABLE 1.1 TEST SPECIMENS

Specimen	Type of Joint Key	Epoxy	Shear Carrying Mechanism	Practical Application
1	No-Key	No	Friction	No
2	Single Key	No	Friction + Key	No
3	Multiple Keys	No	Friction + Keys	Yes
4	No-Key	Yes	Friction + Epoxy Bonding	Yes
5	Single Key	Yes	Friction + Key + Epoxy Bonding	Yes
6	Multiple Keys	Yes	Friction + Keys + Epoxy Bonding	Yes
7	Monolithic Specimen (No-Joint)		$V_c + V_s$	

Fixed Condition $\left\{\begin{array}{l}\text{Concrete Strength}\\ \text{Amount of Prestress}\\ \text{Type of Epoxy}\end{array}\right.$

*Pasco-Kennewick Bridge (-Prestressed concrete cable-stayed bridge with multiple stays-) has no shear keys in the web, but has steel pins in the upper slab.

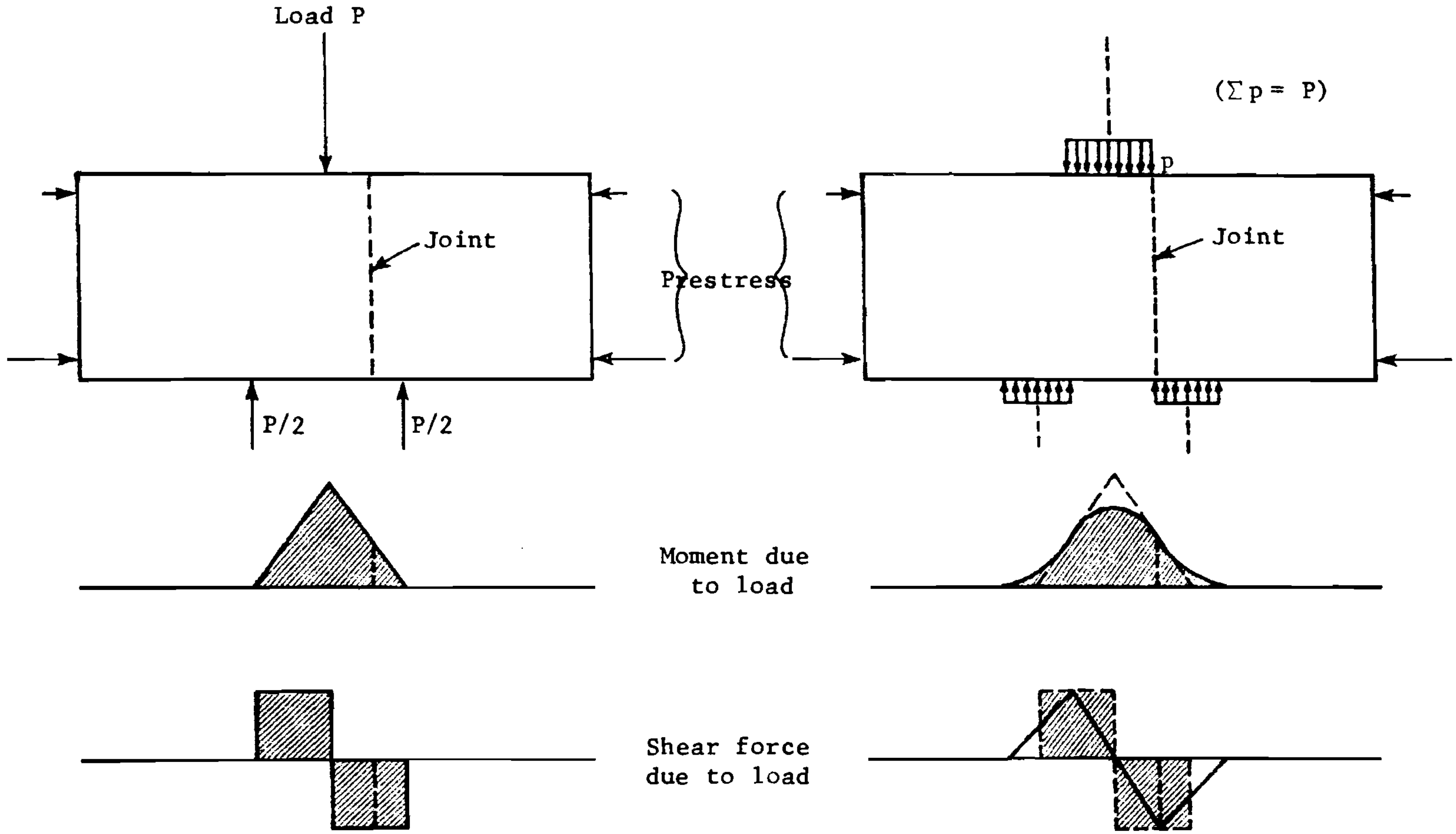


(a) Typical box girder section

(b) Web prototype

(c) 1/4 web model

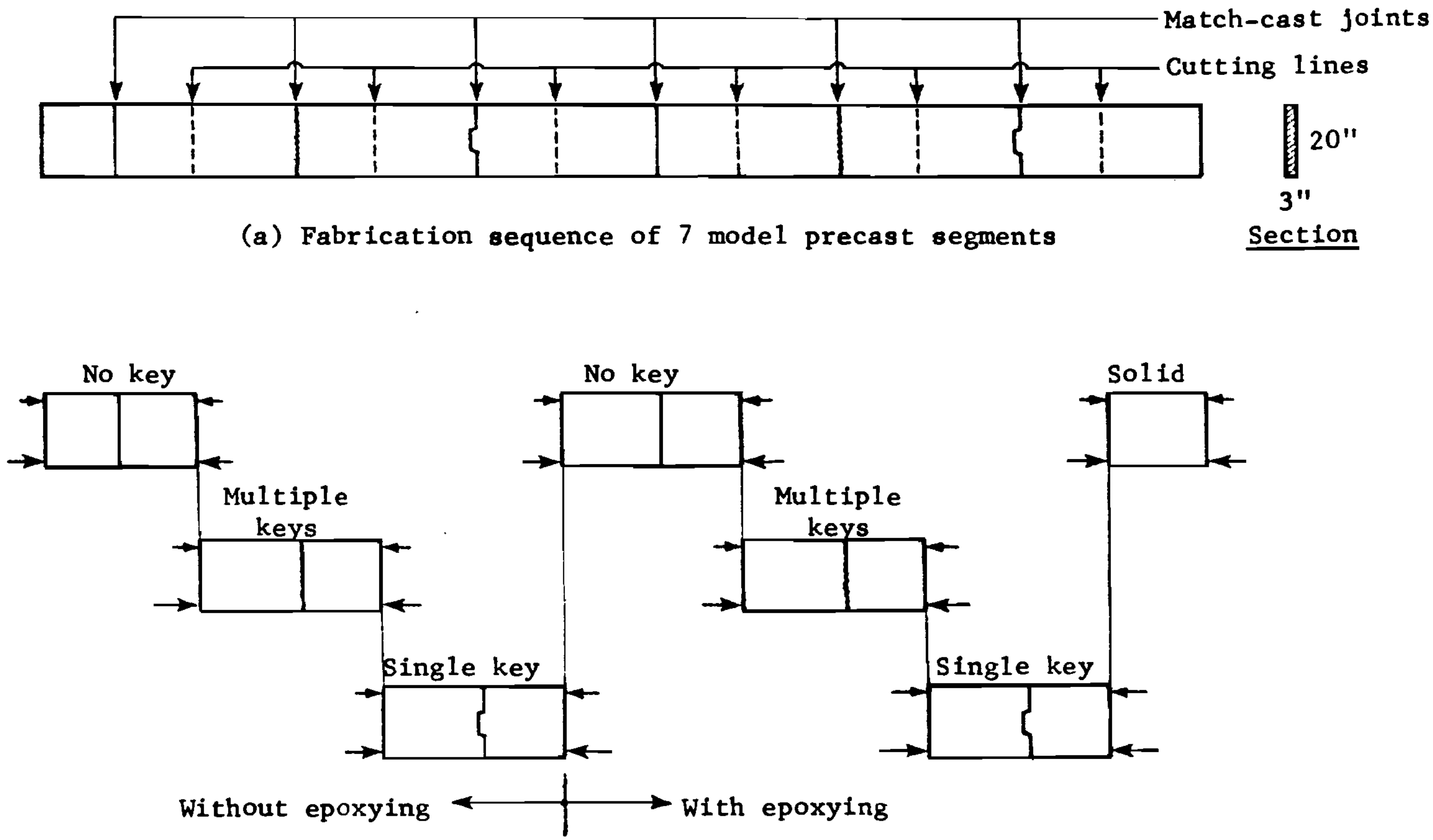
Fig. 1.3 Web model for the tests



(a) Concentrated load application

(b) Distributed load application

Fig. 1.4 Shear test of joint



(a) Fabrication sequence of 7 model precast segments

(b) Test specimens

Fig. 1.5 Model precast segments and test specimens

Technology Corp., Tacoma, Washington. The ACI Building Code [13,14] adopted the theory, referring to other related studies [15,16] by Mattock and Hawkins. The AASHTO Specifications [17] are largely patterned after the ACI recommendations.

Shear friction theory is an explanation of interface shear transfer which states that the shearing force across a potential crack or plane of weakness is resisted by virtue of friction along the crack or plane, if there is a normal compressive force across the crack.

Coefficient of friction values given by Mast, the AASHTO Specifications, the ACI Building Code, and the PCI Design Handbook are as follows:

	Mast	AASHTO and ACI 318-77	PCI Handbook
Concrete placed monolithically	1.4-1.7	1.4	1.4
Concrete placed against hardened concrete with roughened interface	1.4	1.0	1.0
Concrete cast against steel	0.7-1.0	0.7	0.6
Concrete cast against smooth concrete	0.7-1.0	-	0.4

1.3.2 <u>Shear Tests on Joints with No-keys between Precast Post-tensioned Units</u>. A number of such tests have been reported.

1.3.2.1 <u>Test by Franz at the Karlsruhe Technical College in West Germany</u> [18]. The testing method used by Franz is schematically shown in Fig. 1.6. The test result obtained from the specimens without any epoxy, any mortar or indentation in the joint surfaces indicated that the coefficient of friction is practically independent of the amount of the normal force, and the value is around 0.70. It was also independent of the eccentricity of the normal force (existence of bending moment). Applied normal stress at the centroid was 230 to 930 psi.

1.3.2.2 <u>Test by Jones at Cement and Concrete Association in England</u> [19]. The testing method used by Jones is schematically shown in Fig. 1.7. The surfaces of the ends of the specimen butted together

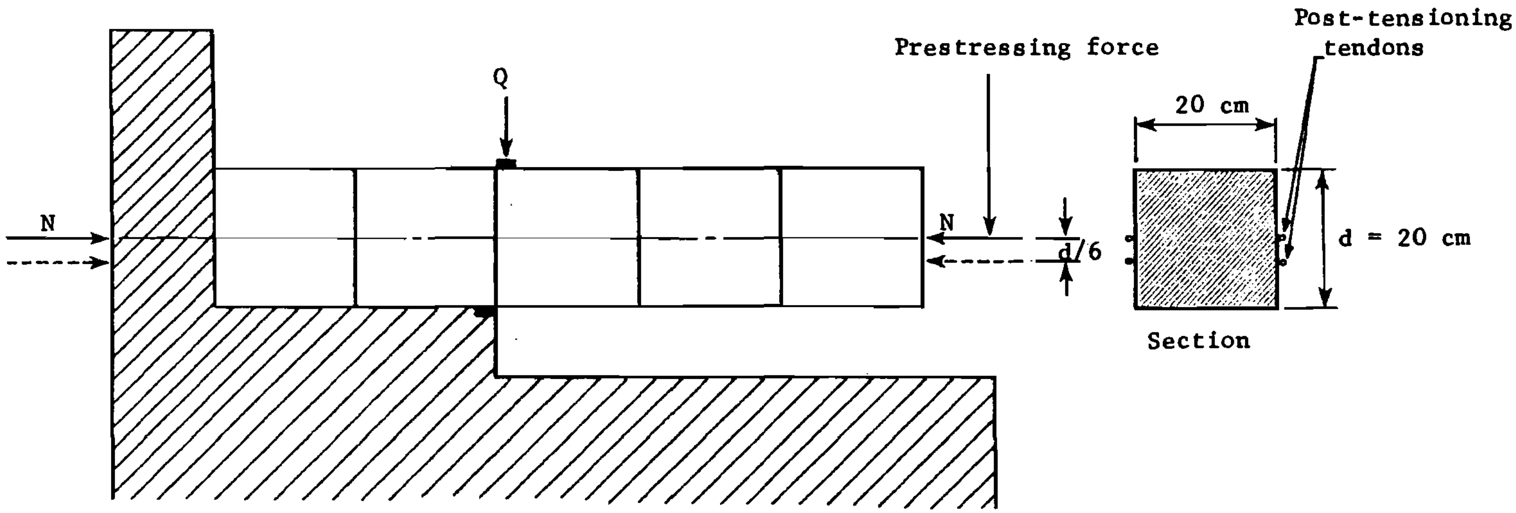


Fig. 1.6 Shear test on joints of precast segmental beams by Franz (Ref. 18)

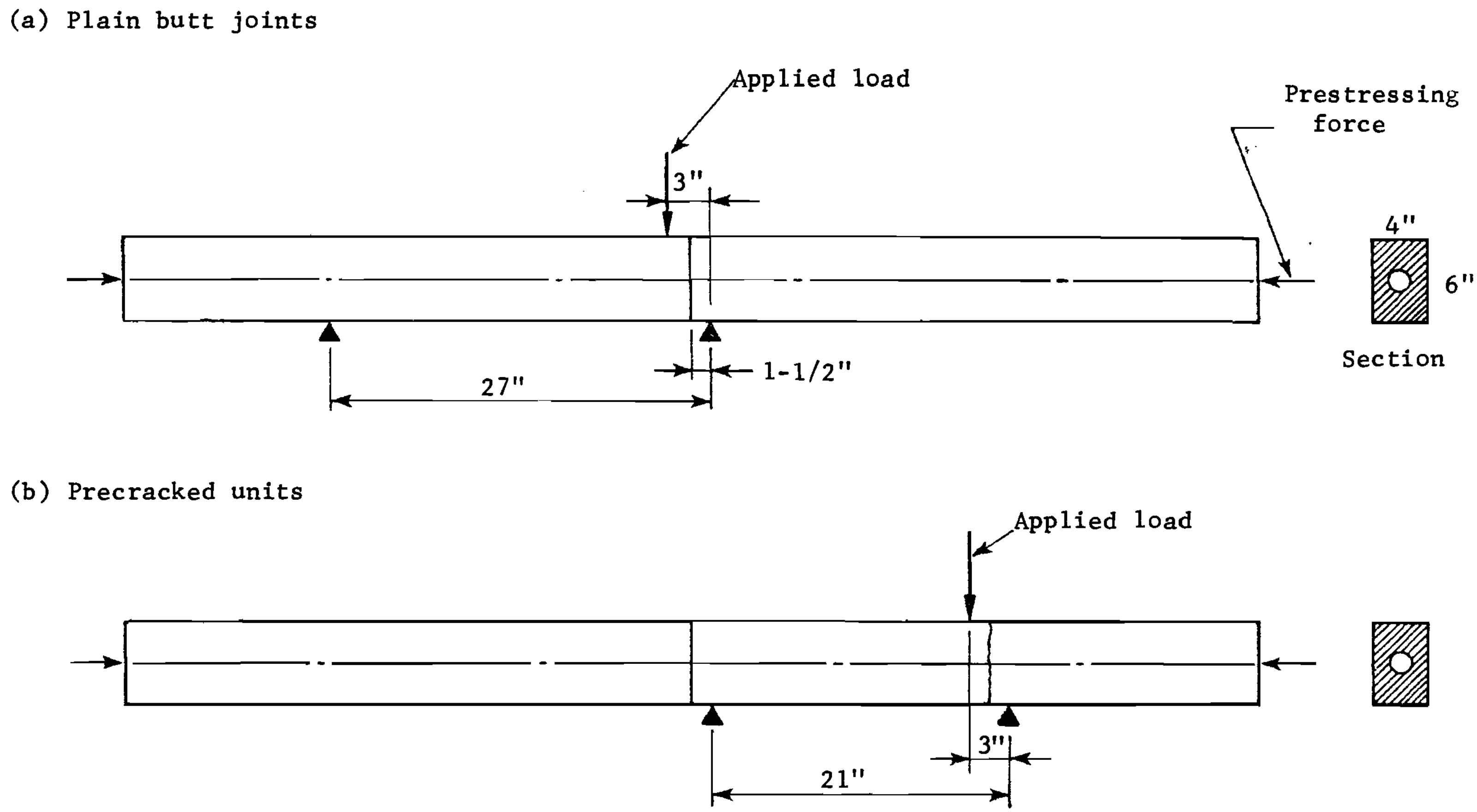


Fig. 1.7 Shear tests on joints between precast units by Jones (Ref. 19)

were obtained by casting against steel bulkheads, thus resulting in a very smooth finish. Failure occurred by slipping of surfaces. The minimum value of the coefficient of friction of the surfaces at the plain butt joint was found to be 0.39 and that of the surfaces at a mortar joint 0.65. Up to 3000 psi of prestress, the coefficient of friction is constant.

1.3.2.3 Test by Gaston and Kriz at Portland Cement Association [20]. The testing method used by Gaston and Kriz is schematically shown in Fig. 1.8. The nature of the contact surface, the contact area of the joint, and the normal stress on the contact area were variables. Half of the specimens were assembled with no bonding medium between the surfaces, and half had a 1-in. layer of mortar between the concrete blocks. The slip between the contact surfaces increased slowly until the maximum load was reached and a sudden, large slip occurred. No visible damage to the contact surfaces of either the bonded or the unbonded specimens was detected. They reported that the coefficient of friction μ may be predicted as

$$\mu = \frac{F}{N} = 0.78 + \frac{43 \times A_j}{N} \quad \text{(for unbonded specimen)}$$

A_j = contact surface area (in.^{2})
N = normal force (lb)

This indicates that μ increases slightly as the contact area increases or as the normal force decreases.

1.3.2.4 Test by Moustafa at Concrete Technology Corp., Washington [21]. The performance of a segmentally constructed prestressed concrete I-beam bridge was investigated by Moustafa. In order to test the joint itself without any help from shear keys or alignment pins, the segments were cast with flat smooth ends. Epoxy was applied on each of the mating surfaces. To determine the shear strength of epoxy joints, small test beams made from 6-in. cubes were prepared in the same way as the segmental girders. The loading was applied in such a way as to force a failure in pure shear at the joints. Failure always occurred in the concrete layer adjacent to the epoxy. The shear strength increased from

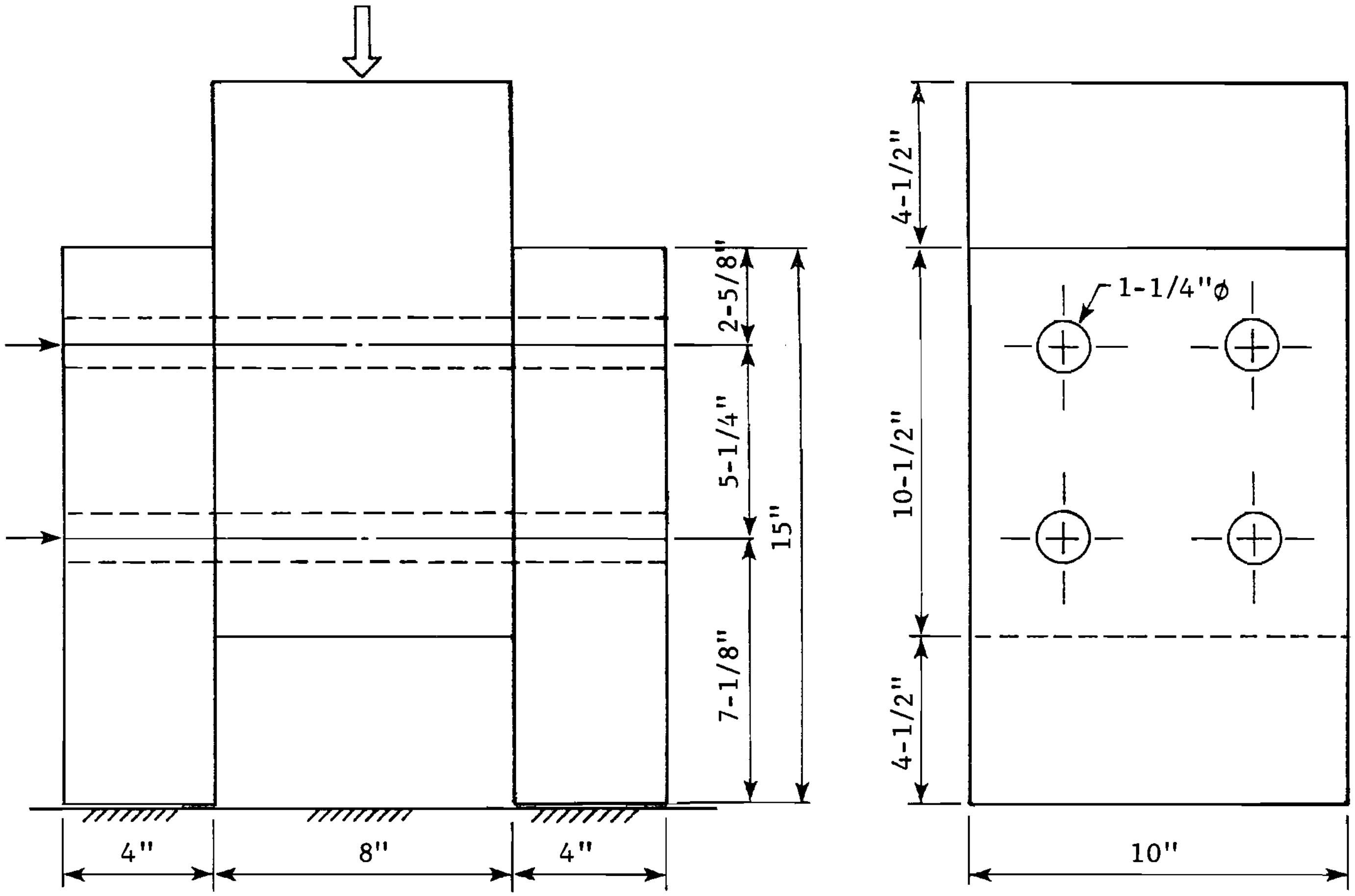


Fig. 1.8 Unbonded push-off test by Gaston & Kriz (Ref. 20)

1130 to 1900 psi when the normal prestress introduced by post-tensioning was increased from 0 to 400 psi. From these results, it was inferred that the shear strength of the joints is not critical in precast segmental beam girders when epoxy is applied to the joints.

1.3.3 Shear Tests on Joints with Single Key between Post-tensioned Precast Segments. Comprehensive studies were made by Kashima and Breen at The University of Texas at Austin [22] related to the JFK Memorial Causeway Bridge in Corpus Christi, Texas. As a part of the studies, an ultimate shear test was carried out, using a 1/6-scale model specimen with whole box girder section and joints having single large keys. Epoxy was applied to the joints. The test specimen essentially developed the theoretical shear for a monolithically cast box girder. The results of the test can be taken as a conclusive indication of the efficiency of properly applied epoxy joints in segmental construction. Provision of these joints did not significantly lower the shear strength of the unit.

Design of the key in single key joints may be considered to be somewhat analogous to that of corbels. Among the references related to the corbel design are Refs. 13, 15, 16, 23, 24, 25, 26, and 27.

1.3.4 Studies on Shear Strength of Multiple Keys

1.3.4.1 MIT Investigation. A comprehensive literature review of the past studies on joints in large panel precast concrete structures was conducted by Zech at Massachusetts Institute of Technology [28]. Several parameters influence reinforced concrete joint strength and behavior. Among the most important of these are the geometry of panel edges, bond between joint and panel concrete, and existence of normal forces simultaneously acting with shear.

(a) Geometry of panel. The geometry of panel edges will determine the amount of mechanical interlock at the joint. In increasing order of strength, these joints may be plain, grooved, or keyed (lightly-heavily). Under monotonic load, keyed joints may be as much as 3 to 4.5 times stronger in ultimate strength than plain ones when the joints are otherwise identically constructed. Strength is dependent not only on the presence of keys but on their shape and size. Tests of sinusoidal and triangular keys have shown less shear strength than the typical trapezoidal forms. It has further been shown that the slope of the key faces should be greater than 55° to 60° for greatest strength. It is recommended that the depth of keys be no less than 0.4 in. and the depth-to-length ratio d/h be greater than 0.125 (see Fig. 1.9). For comparison, examples of multiple key configuration used in three recent segmental bridges are shown in Table 1.2.

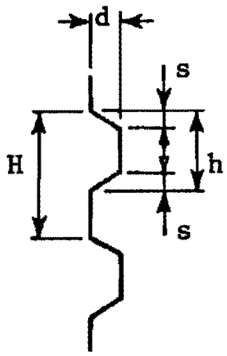
The reason for the depth-to-length ratio limit is that extremely long keys will fail by shearing off or crushing of only one corner, rather than the whole key, with a resulting smaller failure load. In general, the larger the proportion of key area to panel edge area, the stronger the joint will be. This relationship will only be true up to a certain point. Beyond this, failure will commence in the overall panel rather than in the joint keys.

(b) Bond between joint and panel concrete. Until the bond is broken, the behavior of a joint panel assembly is approximately monolithic. Unreinforced joints will fail by slip at the contact surface as soon as bond is broken. This is a distinctly brittle failure. In one case, shear capacity of unbonded castellated joints was found to be higher. This was thought to be due to the more uniform shear distribution along the joint.

(c) Existence of normal compressive forces. The effect of normal compression is cumulative with that of the shear friction steel across the joint, so that the important parameter was found to be $N + A_v f_y$. This is reasonable since the effect of either is to increase frictional resistance. The effect of normal compression is the same as that of the clamping action of the reinforcement, except that slip is required to mobilize resistance of the shear friction reinforcement. Post-tensioned joints provide an added shear friction resistance beyond that in an otherwise identical nonprestressed joint.

1.3.4.2 Test by Kupfer, Güchenberger, and Daschner at the Technical University, Munich [29]. A very interesting series of experimental investigations to determine the structural behavior of segmental precast prestressed girders with cement mortar joints and epoxy bonded joints was undertaken by Kupfer et al. In preliminary tests, both the use of a modified cement mortar by application to very

TABLE 1.2 EXAMPLES OF MULTIPLE KEY CONFIGURATION



Bridge	Long Key	Red River	Linn Cove
Number of Keys	9	7-31	12
h/H	.60	.73	.70
d/s	2/1	1/1	1.25/1
d/h	.32	.31	.36

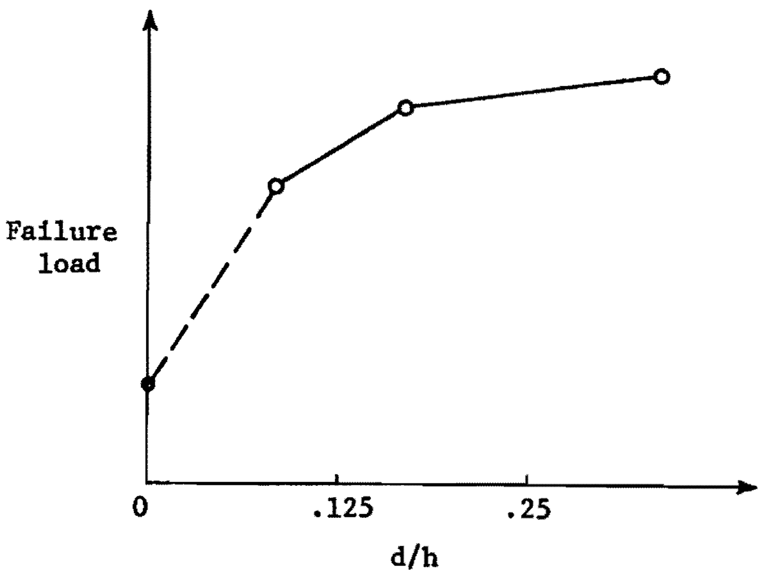


Fig. 1.9 Effect of shear key height to depth ratio (Ref. 28)

thin joints of only a few millimeters wide (closed joints) and the development of the strength (green strength) of grouted joints of a few centimeters wide were investigated. Concrete prisms with joints oriented obliquely to the direction of compression (see Fig. 1.10) may be regarded as sections of a compressing strut in the joint area of a segmental precast girder. Compression tests on such prisms with cement mortar joints provided with multiple keys and a large angle ($\alpha = 50^{o}$) yielded a joint strength amounting to 91% of that of comparable monolithic concrete prisms in the case of the closed joint and 78% in the case of the grouted joint . The structural behavior (cracking) and the load capacity (especially concerning transverse shear) were further investigated by load tests on two segmentally constructed prestressed girders, about 9 m long and 70 cm deep (see Fig. 1.11). The girder with multiple key joints of modified cement mortar showed the same favorable behavior as the girder with epoxy bonded multiple key joints. The web failing in inclined compression indicates that in both cases the load capacity due to shear was not impaired by the presence of the multiple key joints.

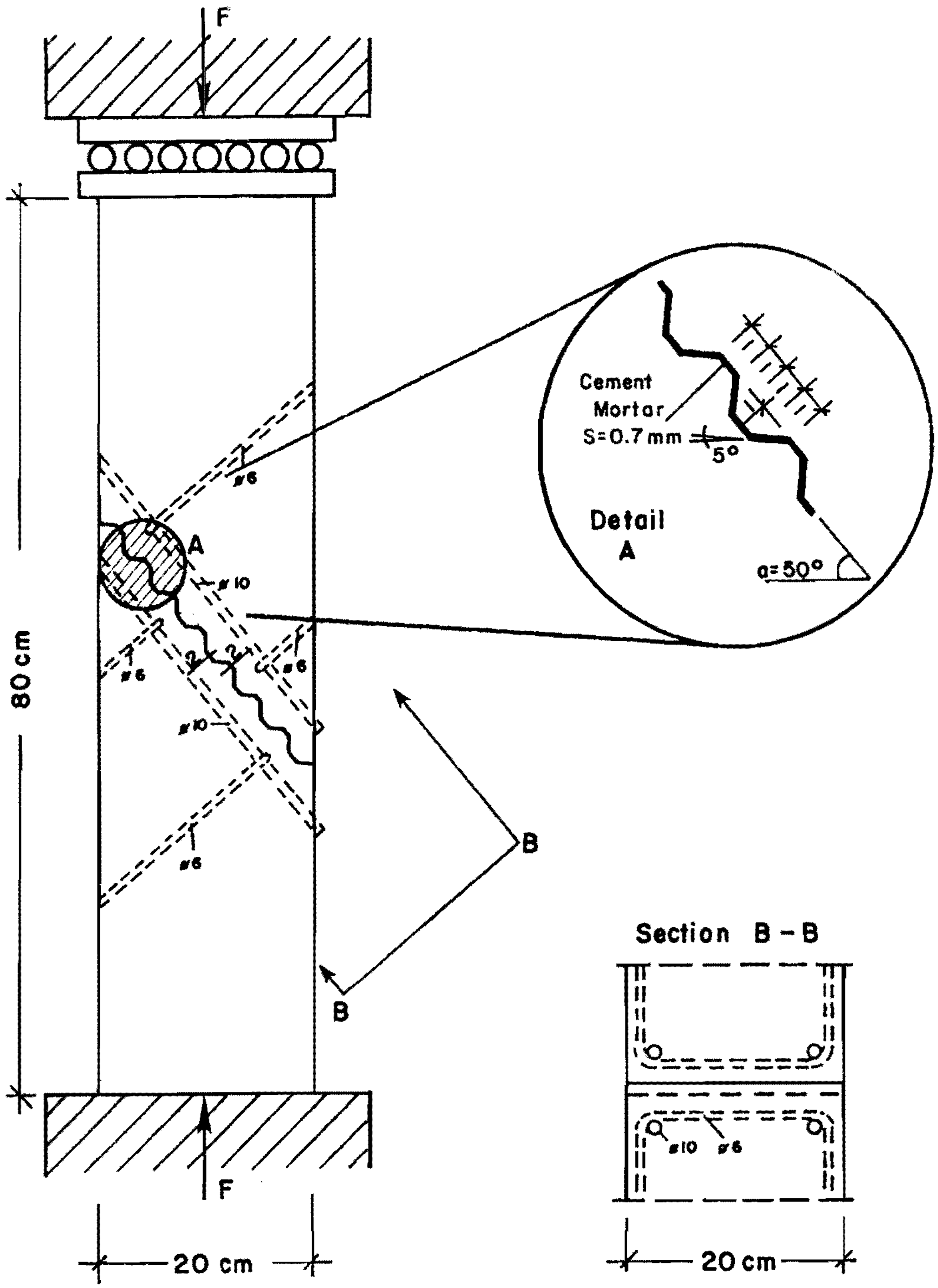
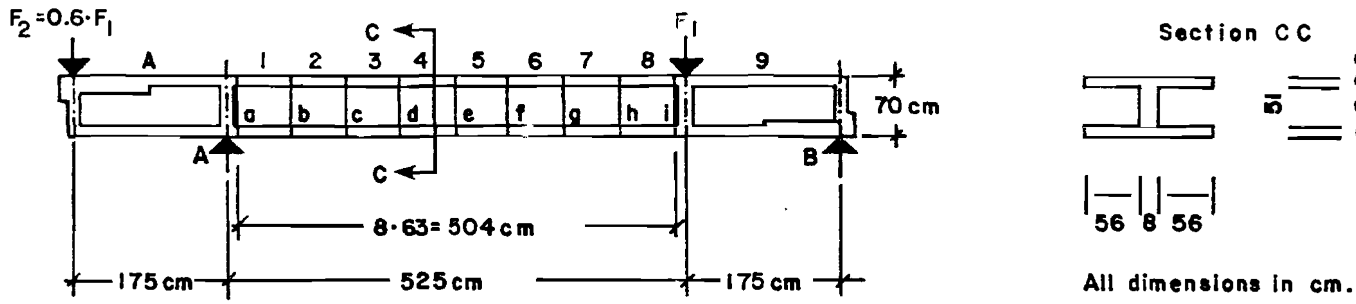


Fig. 1.10 Concrete prism specimen (from Ref. 29)



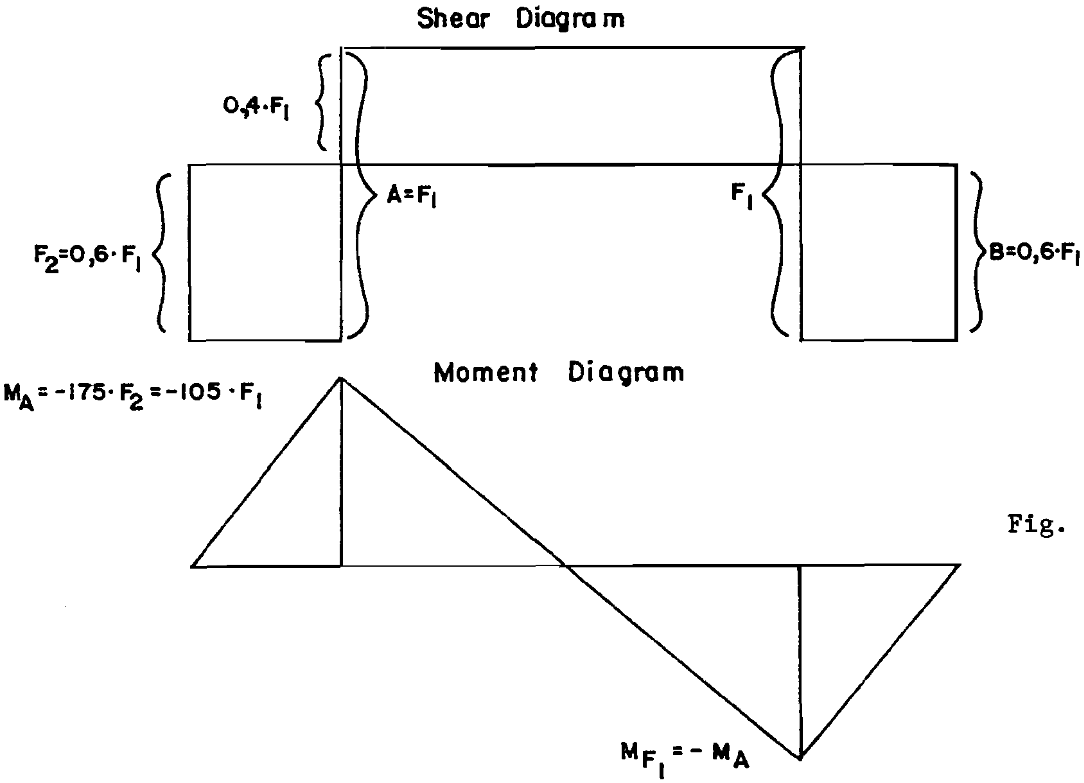


Fig. 1.11 Segmental girder specimen (Ref. 29)

CHAPTER 2

TEST SPECIMEN

2.1 Specimen Dimensions

Figure 2.1 shows the general profile of the model precast segment. Dimensions of the segments were 3 in. (width) x 20 in. (depth) x 48 in. (length), except for the first segment whose length was 20 in.

For stirrups, 6 mm (1/4 in.) deformed bars were used at 3 in. spacing. Ten gage wire was also used at 3 in. spacing for supplementary longitudinal reinforcement. The reinforcement cage of this 1/4 scale model segment corresponds to typical prototype web reinforcement of #8 stirrups and #4 longitudinal bars both at 12 in. spacing (see Fig. 1.3). However, this correspondence of model reinforcement to that of some prototypes has relatively little importance since the joint shear test (shown in Fig. 1.4b) was chosen so that the maximum shear force occurs only in the joint vicinity or at the mirror image of that joint. However, the web reinforcement is important in the test of the monolithic model with no joint used as the baseline for comparison of the various jointing methods.

Two tendon ducts (one at the top and the other at the bottom) were placed in each specimen with 8 in. of eccentricity. Prestressing tendons were later inserted to provide normal force and bending moment resistance. The duct location caused minimal disturbance in the center portion of the joint surface and the tendons prevented significant flexural tensile stresses in the specimen.

The three types of joint configurations examined are illustrated in Fig. 2.2. In the single key specimens, both male and female keys were reinforced with 10 gage wire. On the other hand, in the multiple key segments, no reinforcement in the keys was provided as is the practice in usual multiple key joint construction. A trapezoidal shape without any intentional rounding off of the corners was used for the multiple key configuration. The characteristics of the single and multiple keys were meant to be similar to those of the JFK Memorial Causeway Bridge and the Long Key Bridge, respectively.

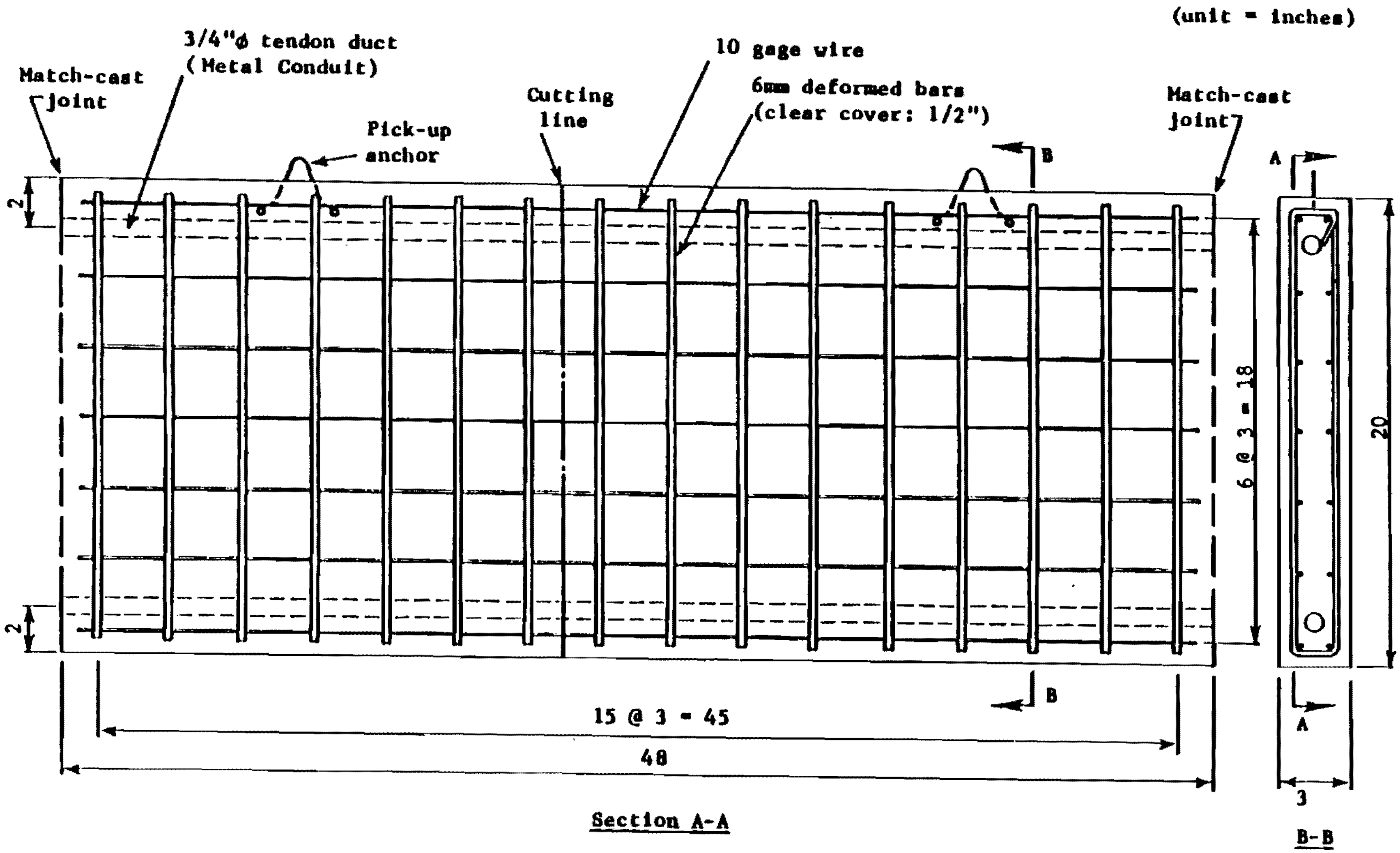


Fig. 2.1 Profile of model precast segment

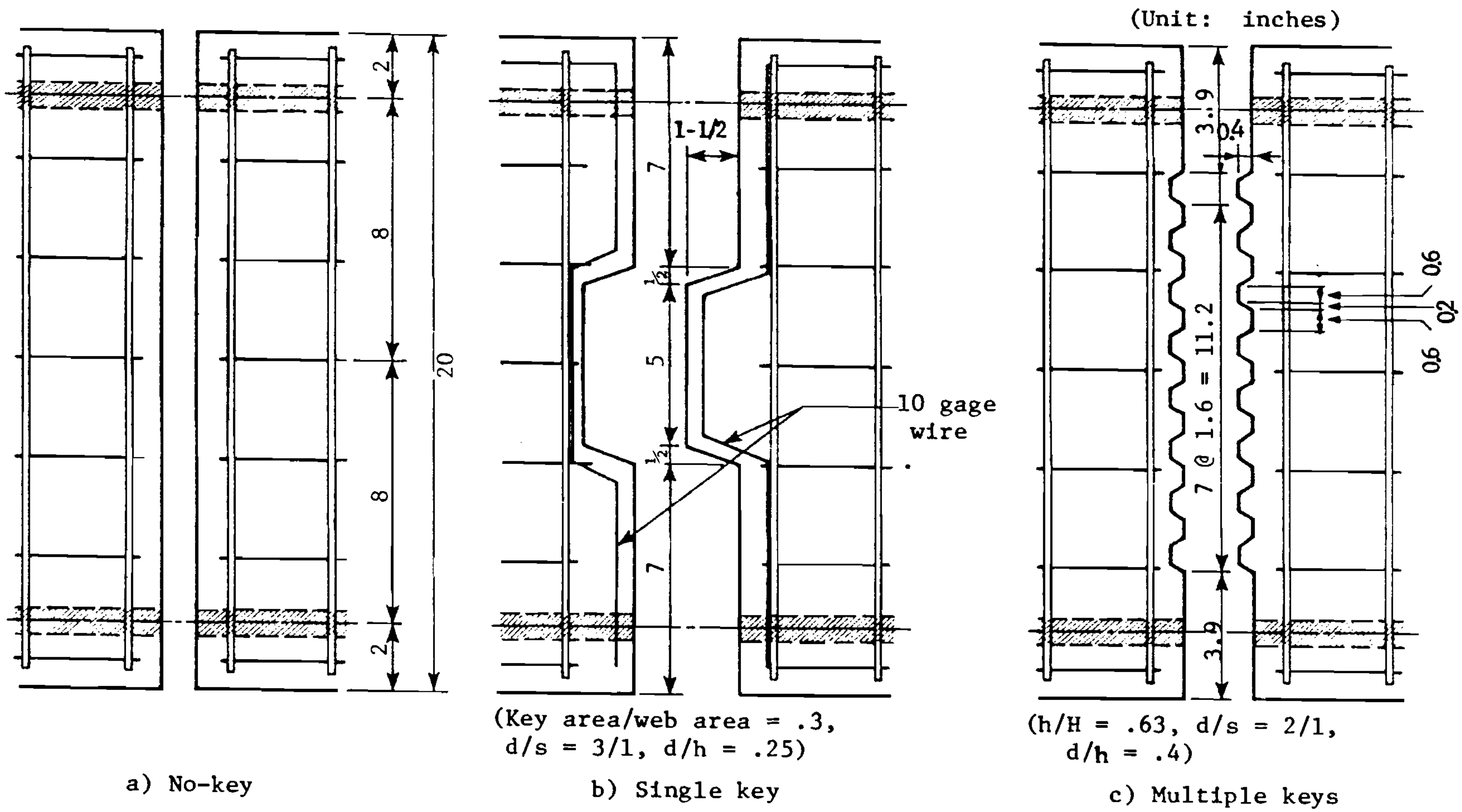


Fig. 2.2 Configuration of joints

2.2 Materials

2.2.1 Microconcrete. Microconcrete was used as a model concrete. Mix proportions of the microconcrete were determined based upon that used by Stone [10]. The basic design mix proportion used is shown in Table 2.1. Design 7-day compressive strength of the concrete was 4000 psi. Twelve 3 in. diameter x 6 in. cylinders were made at each casting of concrete. Cylinder specimens were tested at 7 days and at the time of each shear test. Concrete strengths at time of testing are shown in Table 3.2. "Prototype" for the concrete mix proportion was "Type H" specified by the Texas State Department of of Highways and Public Transportation [10].

2.2.2 Reinforcement. Six mm (1/4 in.) Grade 60 deformed bars and 10 gage plain annealed wire were used as reinforcement. Figure 2.3 shows a stress-strain relationship of 6 mm rebar. The mechanical properties of the bars are also shown in the figure. The stress-strain relationship of the annealed wire is shown in Fig. 2.4. Yield strength of the wire was much lower than expected. The wire was not deformed, but several studies verified that the bond strength of the wire to microconcrete would be adequate.

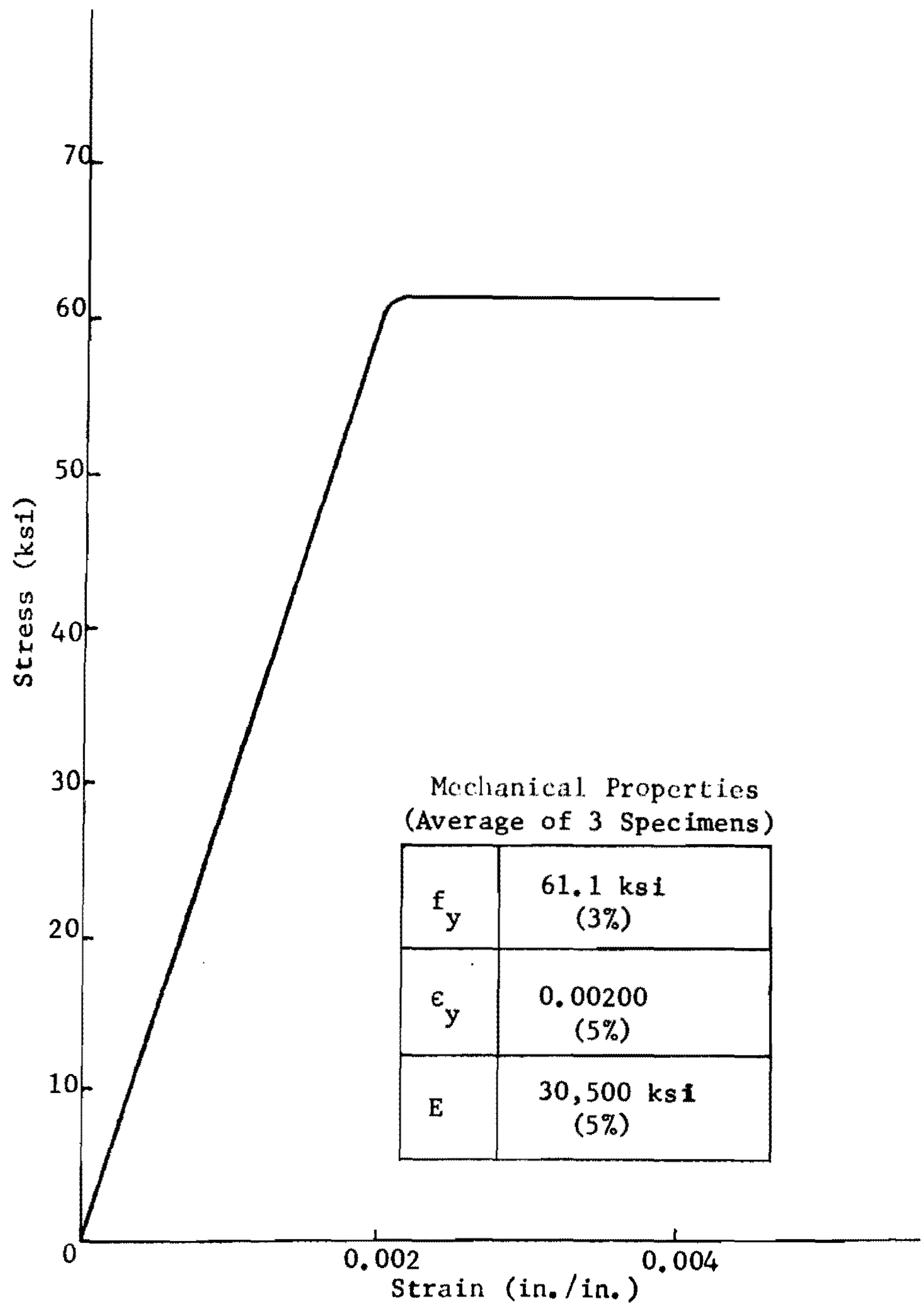
Although, as mentioned in Section 2.1, reinforcement of the specimens might not have a significant meaning in the shear test except for the baseline monolithic specimen, such conventional practice in making structural model test specimens was followed in all specimens.

2.2.3 Prestressing Tendon. Two-hundred seventy ksi, 1/2 in. diameter seven-wire prestressing strand (f_{pu} = 270 ksi, A_{ps} = 0.153 in.^{2}) was used in the test. The tendon sheath was flexible metal conduit with 3/4 in. inner diameter.

2.2.4 Epoxy. The epoxy bonding agent was a standard concrete patching adhesive formulated by the Texas State Department of Highways and Public Transportation (Epoxy Adhesive A103). The strength development characteristics of the epoxy are shown in Fig. 2.5. These strength development curves were obtained from a simple bond test using a metal-to-metal lap shear joint, which was carried out using 2 in. wide metal sheet strips. This preliminary test was conducted with two different lap areas, i.e., one with 2 in. x 1/2 in. lapping and the

TABLE 2.1 DESIGN MIX PROPORTION (per ft^3)

3/8" aggregate	41.0 lbs
#1 blast sand	33.8 lbs
Ottawa silica sand	36.8 lbs
Type III cement	21.25 lbs
Water	14.8 lbs
Admixture (ASTM C494 Type B)	0.51 fl. oz.
Water/cement ratio	0.70
Cement ratio	6.4 sacks/yd^3



f_y	61.1 ksi (3%)
ϵ_y	0.00200 (5%)
E	30,500 ksi (5%)

Fig. 2.3 Stress-strain relationship of 6mm reinforcing bars

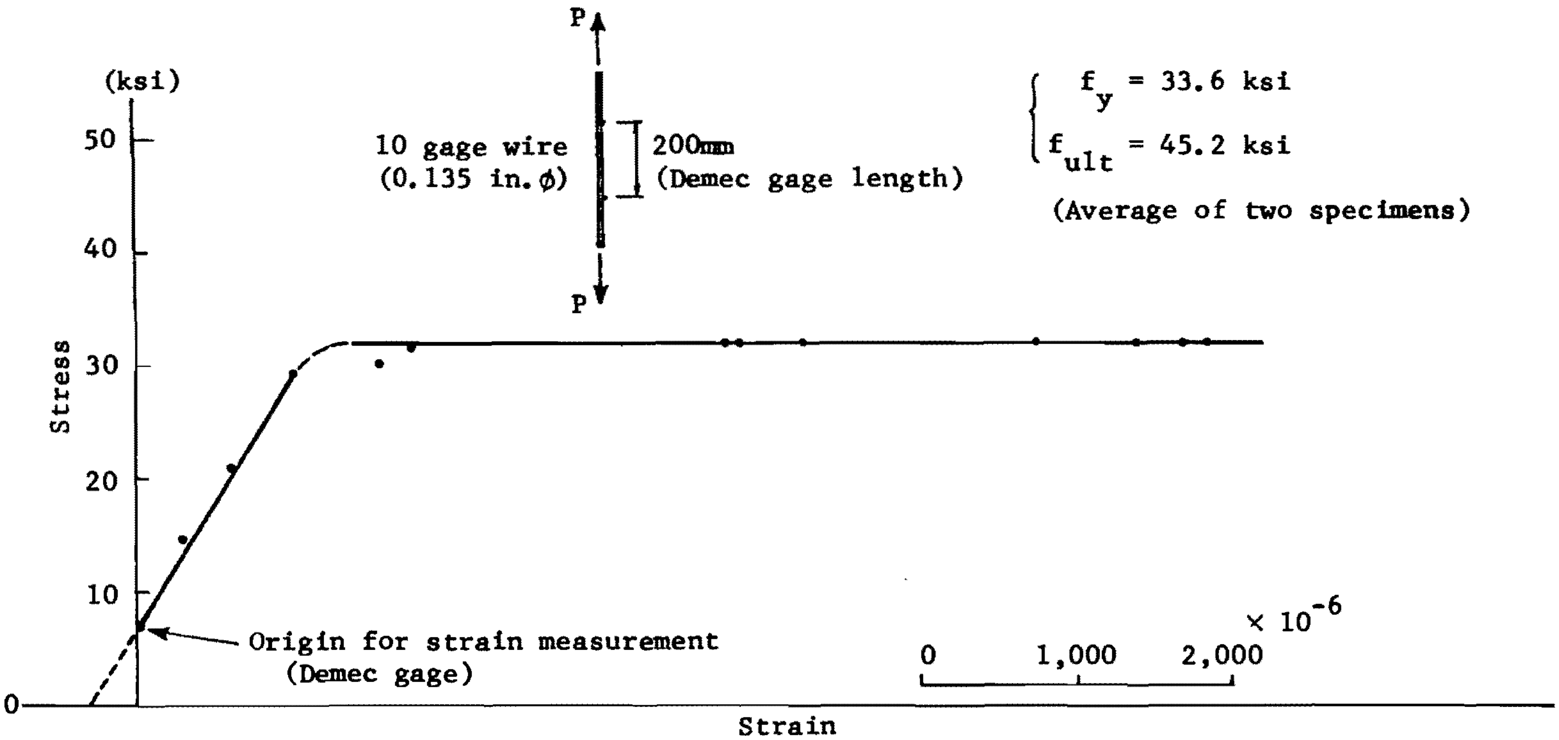


Fig. 2.4 Stress-strain relationship of 10 gage wire

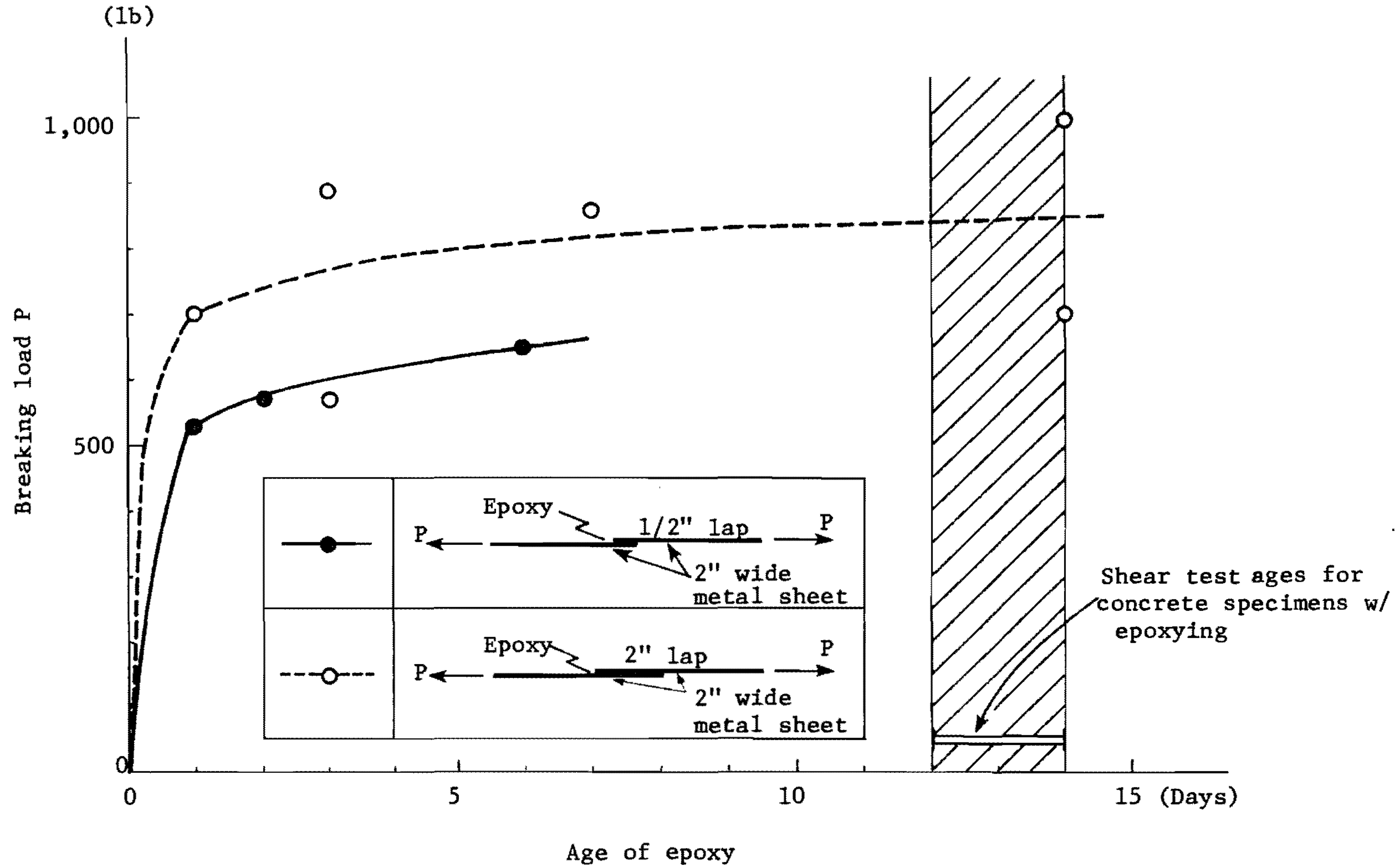


Fig. 2.5 Development of strength of epoxy

other with 2 in. x 2 in. As seen in Fig. 2.5, both specimens gave similar development curves of epoxy strength. The shear test for the epoxied concrete specimens was planned to be carried out when the epoxy bonding agent was considered to have developed its strength sufficiently. The epoxy bonding agent was exposed to the same laboratory curing conditions (mainly, temperature), both in the preliminary test on the metal strip specimen and in the shear test of the concrete web model specimens.

The compressive strength of the epoxy was approximately 5000 psi at 24 hours according to a rough compression test. It was assumed that the strength would reach more than 6000 psi at the time of the shear test.

The following is an extract from "Texas Highway Department Special Specification Item 2131 Epoxy Bonding Agent," which is based upon the work done by Kashima and Breen [20] at The University of Texas at Austin:

> The epoxy material shall be of two components, a resin and a hardener, meeting the following requirements:
>
> | | | |
> |---|---|---|
> | a. | Pot life | 90 minutes min. at 68°F |
> | b. | Compressive strength | 6000 psi min. |
> | c. | Tensile strength | 2000 psi min. |
> | d. | Specific gravity | 70 to 120 lbs/cu. ft. |
> | e. | Viscosity at 68°F | 10,000to 50,000 cps |
> | f. | Coefficient of thermal expansion | Within 10% of that for concrete |
>
> The joint material shall be able to develop 95% of the flexural tensile strength and 70% of the shear strength of a monolithic test specimen.

The <u>Precast Segmental Box Girder Bridge Manual</u> [2] specifies seven epoxy bonding agent tests, which are (1) sag flow, (2) gel time, (3) open time of mixed epoxy bonding agent, (4) three-point tensile bending test, (5) compression strength of cured epoxy bonding agent, (6) temperature deflection of epoxy bonding agent, and (7) compression and shear strength of cured epoxy bonding agent.

These tests on the epoxy were not performed in this study since this epoxy had been examined thoroughly in connection with other projects [22,30].

2.3 Fabrication

2.3.1 Reinforcing Cages and Forms. The cages were tied by hand and shear key reinforcement was added for segments with single keys.

Plywood forms were used. In order to firmly position the preceding segment in match-casting, an adjacent bottom form panel was provided. Three types of end forms were fabricated. These were for the segments with no-key, with single key, and with multiple keys, respectively. Single and multiple key configurations were produced using metal sheet, as shown in Fig. 2.6.

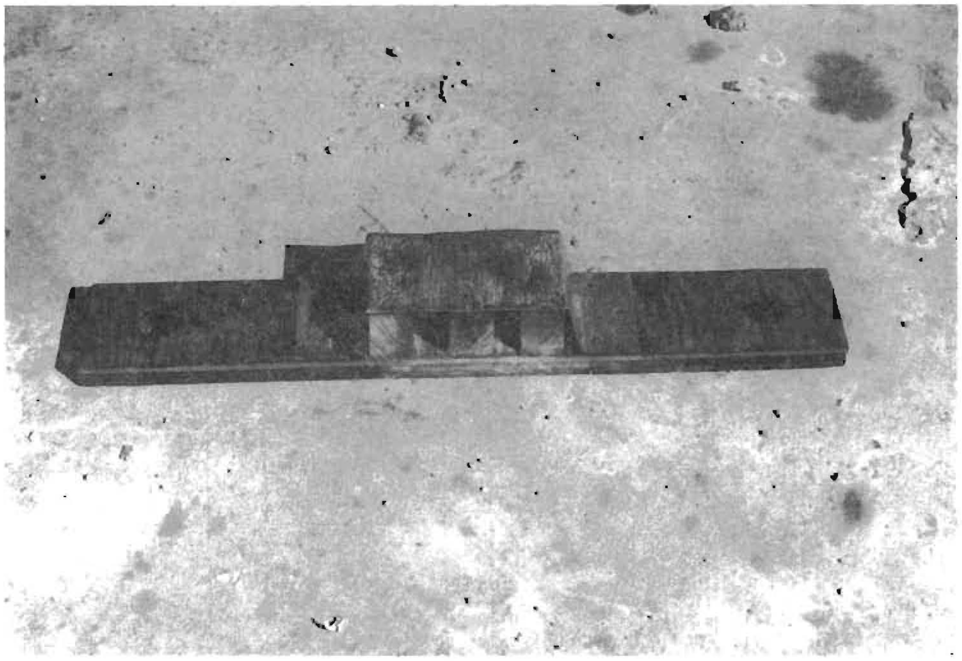
2.3.2 Casting Procedure. Concrete segments were carefully fabricated. Special procedures included:

Preparation

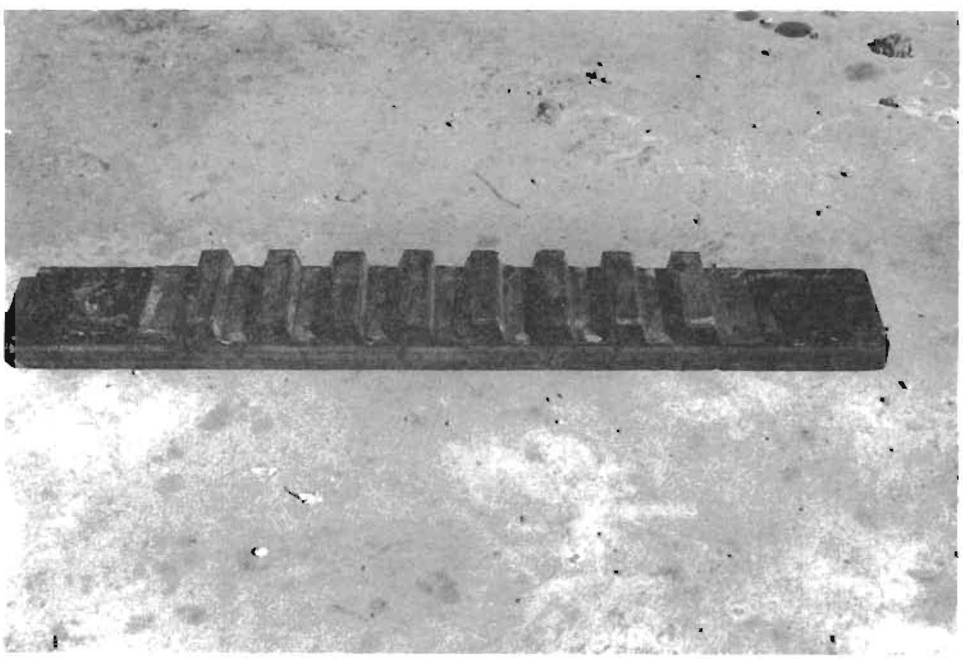
--Lining of the end face of the previously cast segment with aluminum foil for bonding breaking purposes. (This was done successfully. There was no trouble in lining even multiple shear keys. The aluminum foil was also painted with lacquer and oil so as to avoid chemical reaction between the aluminum and the high alkaline cement paste solution. In actual match-casting of precast segments, a plain vegetable soap, sometimes mixed with talcum powder, is often used as a bond breaker between segments. It is rinsed off with water after stripping, since the face of the segments must be clean for application of the epoxy. Gallaway [31] reports that to aid in stripping and provide a small clearance during erection, the female portion of the previously cast shear key can be lined with plastic tape. The use of aluminum foil eliminated the rinsing-out operation of the vegetable soap.)

--Assemblage of the form panels in match casting position (Fig. 2.7).

--Insertion of temporary steel rods in tendon sheaths to keep the sheaths straight.



(a) Single key



(b) Multiple keys

Fig. 2.6 End forms

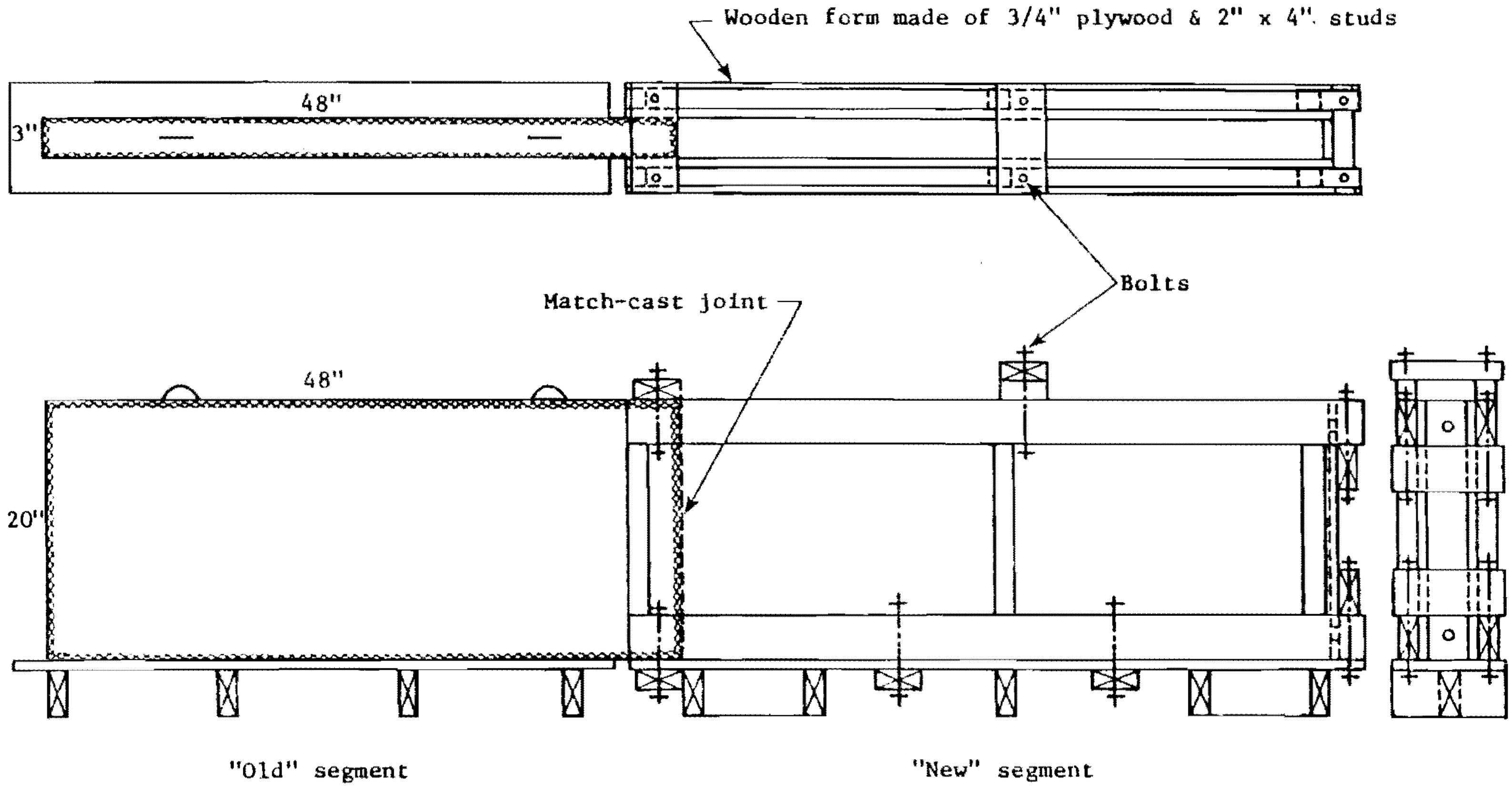


Fig. 2.7 Match-casting of segments

Casting and Curing

--Microconcrete was mixed in a 2 cu. ft. laboratory mixer.

--Concrete was compacted internally with 6 mm steel rods, and externally with a concrete vibrator.

--Forms were removed 24 hours after casting.

--Separation of the segments (Fig. 2.8).

--Cutting and grinding of sheath at end face with a hand saw and a hand grinder.

--Movement of the segment into the next position for match-casting.

--Curing of segment concrete and cylinders with plastic sheet cover and occasional water supply for approximately 1-2 months.

2.3.3 Concrete Strength f'_{c7}. ϕ3 in. x 6 in. cylinder specimens were capped with sulfur capping material. Three cylinders were tested for each concrete at 7 days after casting for the purpose of quality control. The test results for the 7 day compressive strength of the concrete are summarized in Table 2.2. Overall average of f'_{c7} was 4470 psi and the coefficient of variance of the data was 8%. This result was considered to be satisfactory.

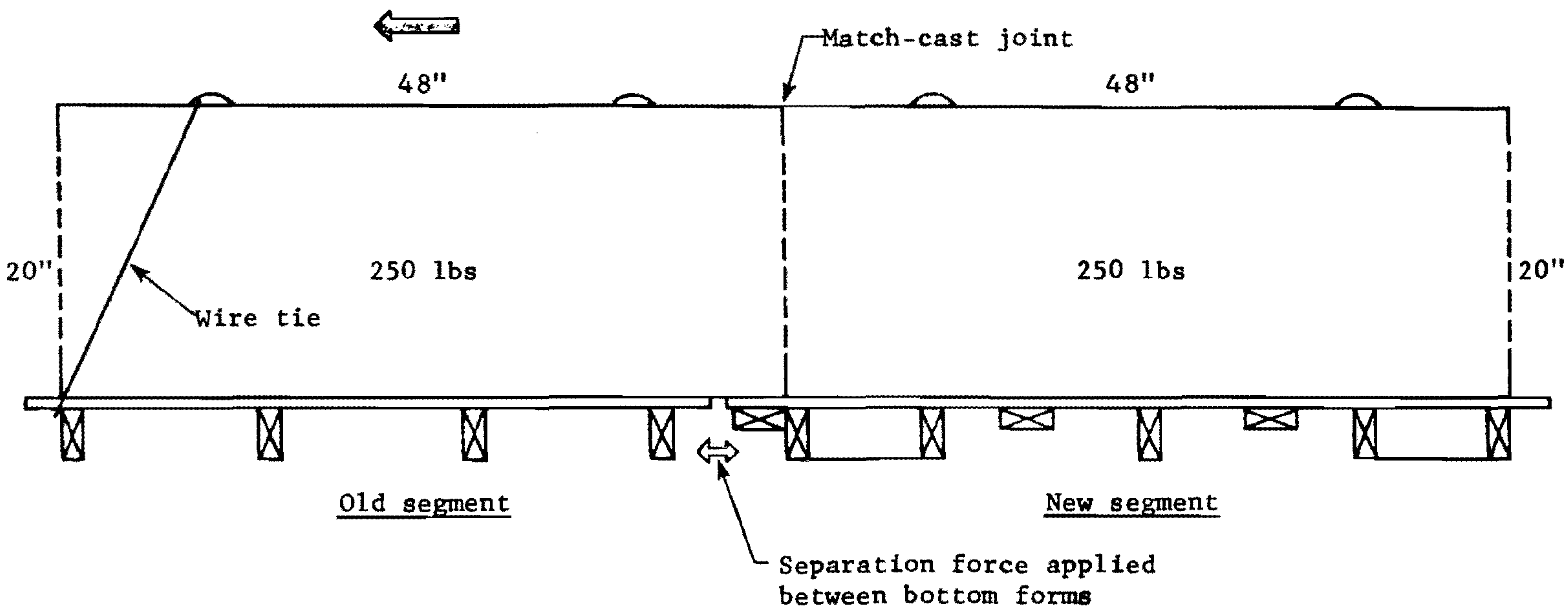
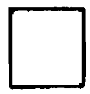
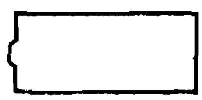
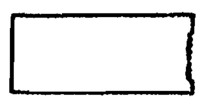
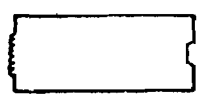
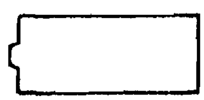


Fig. 2.8 Separation of segments after form removal

TABLE 2.2 CONCRETE STRENGTH AT 7 DAYS

Casting Sequence	Segment	f'_{c7}(psi) (Coefficient of variance) Each (Ave. of 3-ϕ3"×6")	Total
1		4,670 (7%)	
2		5,120 (4%)	
3		4,080 (5%)	
4		4,240 (5%)	4,470 (8%)
5		4,360 (7%)	
6		4,340 (5%)	
7		4,480 (1%)	

CHAPTER 3

TEST PROCEDURE AND RESULTS

3.1 Preparation

Each model segment was cut into two parts, as illustrated in Fig. 1.5, using a concrete saw. The cut surfaces were the exposed end faces of the test specimens and were not to be joined.

3.2 Joining and Post-Tensioning

3.2.1 Match-cast Joint Surfaces. For match-cast joints, the surface including the formed keys should be even and smooth to avoid point contact and surface crushing or chipping off of edges during post-tensioning. It is particularly important before applying epoxy that the adjacent surfaces are solid, clean, and free of dust and greasy materials. As in any adhesive bonding, the preparation of the surface will quite often determine the success of the joint.

Tiny pits were observed in some of the match-cast keyed joint surfaces. However, they were left as they were, since none of them seemed to be harmful even for the specimens tested without epoxy. Such pits can be found in the surfaces of actual match-cast precast segments.

The joint surfaces were cleaned with a wire brush and wiped with acetone. In actual precast segmental construction, it is recommended that light sandblasting be used for preparation of the concrete surfaces for good bonding.

3.2.2 Epoxying. Components of the epoxy mix (resin and hardener) were proportioned and mixed thoroughly until a uniform color was obtained, following the instructions of the manufacturer.

The concrete surfaces to be bonded were kept dry. The epoxy adhesive was applied by hand, using protective gloves, immediately after mixing. Both mating surfaces were coated and brought together while the epoxy was still viscous.

The Prestressed Concrete Institute [2,32] specifies that a minimum compression of 30 psi shall be provided by means of temporary post-tensioning over the entire joint area during the open time period of the bonding agent until the permanent tendons are stressed. Uniform post-tensioning stress of about 70 psi was applied in this series. During the provisional post-tensioning, excess epoxy was squeezed out of the joint. Care was taken to prevent epoxy from entering the tendon ducts. The temporary clamping force was provided using the post-tensioning system described in Sec. 3.2.3. The load was controlled by calibrated load cells and pressure indicators for the first specimen. After obtaining the relationship between load cells and pressure indicators, only the latter were used. The epoxy joints were air-cured inside the laboratory for 12 to 14 days before the specimens were tested.

3.2.3 Prestressing. The post-tensioning arrangement is shown in Fig. 3.1. Special provisions in the post-tensioning procedure included:

Preparation - Large end plates were used to distribute the bearing stresses to prevent end surfaces from bursting or chipping off.

Prestressing - As seen in Fig. 3.1, anchoring chucks with nuts and anchor plates with slits were used for the purpose of detensioning and removal of the strands after the test.

- Stepwise loading of prestress so that significant tensile stress would not appear in any part of the specimen concrete. Top and bottom strands were tensioned alternately. The loads were monitored by load cells along with pressure indicators. Relatively large seating loss was experienced. This was partly because the length of the specimens, which was 48 in., was short. The amount of prestress was determined as described in Sec. 3.2.4.

3.2.4 Prestressing Forces. According to the ACI Building Code [13], maximum permissible tensile stress in prestressing tendons due to jacking force is $0.8f_{pu}$. In the test, the bottom strand was tensioned up to $0.7f_{pu}$, which corresponds to 28.9 kips. No attempts were made to increase this value. As mentioned above, significant amounts of seating loss were observed after force transfer. The force in the top tendon was adjusted to maintain the condition that the stress from prestressing alone at the top fiber would be zero.

As will be shown later, the average prestressing forces in the bottom and top tendons just before the shear test were 18.7 kips and 6.7

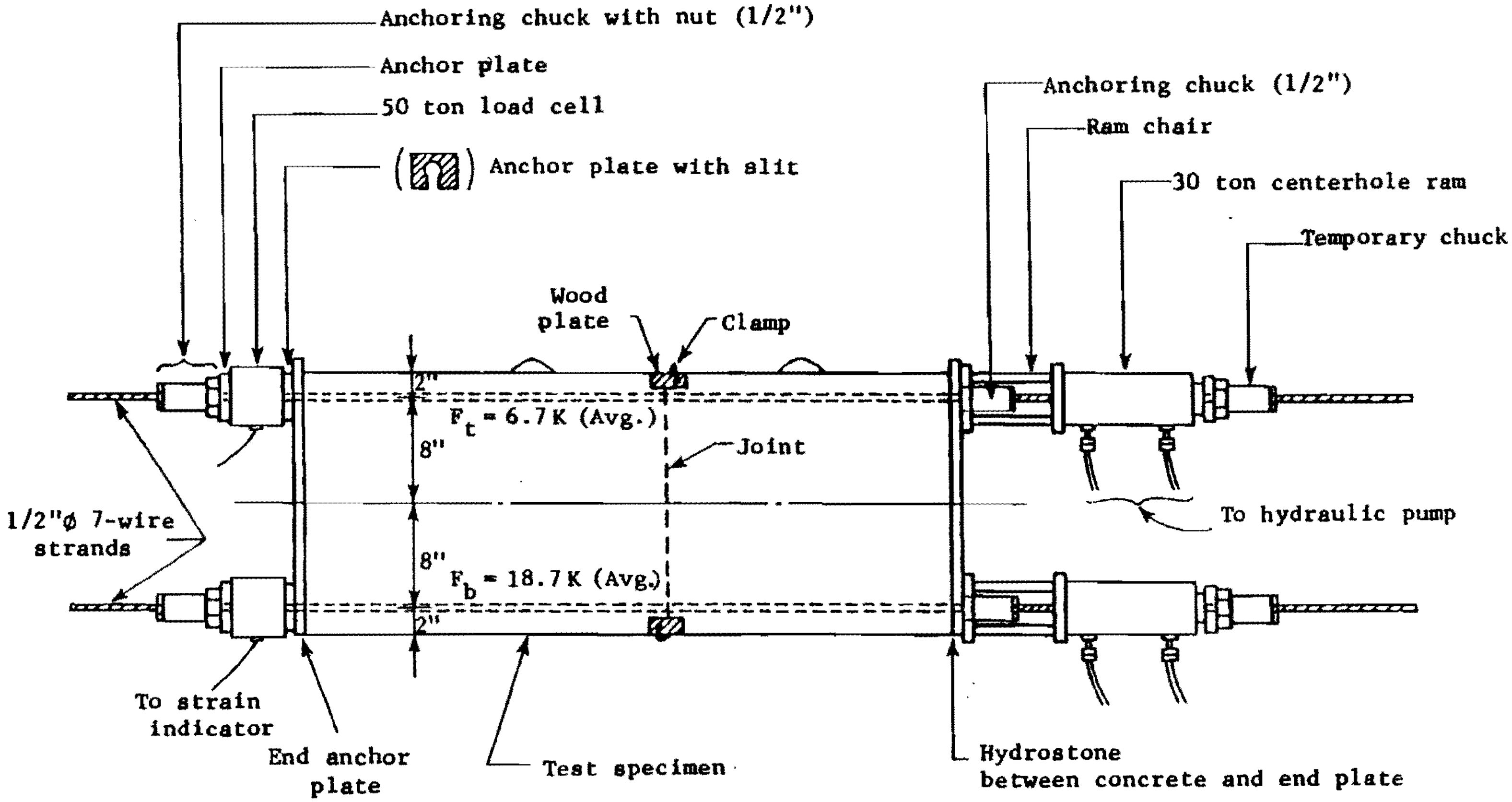


Fig. 3.1 Post-tensioning arrangement

kips, respectively, with a total of 25.4 kips. The average compressive stress in concrete due to the prestressing was 424 psi with the coefficient of variance of 2%.

3.3 Shear Test Arrangement

Figure 3.2 illustrates the shear test arrangement. The load was applied using a calibrated SATEC testing machine. In order to achieve a distributed load application, three sets of bearing plates were used, as seen in Fig. 3.2. Hydrostone was applied between the top loading plate and the top surface of the specimen. Since bottom surfaces of the specimens were as flat as the bottom form panel, no hydrostone was used there. Instead, the bearing stress was lowered by increasing the bearing area of the plate. This worked well and no damages were observed at the bottom surface of the specimen after the test.

The test was conducted immediately after the application of the final prestressing. Tendons were not grouted. During the test, force changes in prestressing strands were measured using load cells. No significant changes in prestressing forces were observed until major slip took place.

Slip at the joints was determined visually and by use of a dial gage. The dial gage was attached to the specimen with a clamp, so that it could indicate a relative displacement between the points across the joint. It should be noted that the original purpose of the dial gage was just to detect the slip occurrence and, therefore, the setup was simple rather than sophisticated.

3.4 Test Results

3.4.1 Stresses Due to Prestressing.

Stresses in each specimen introduced by prestressing are summarized in Table 3.1. Overall averages of prestressing stresses at top, at centroid, and at bottom were 56 psi (tension), 424 psi (compression), and 903 psi (compression), respectively. Although slight flexural tensile stresses were computed in the top portion of the segment section, this was not observed to cause any troubles in the test. Figure 3.3 shows that the test specimens had only minor flexural tensile stresses at the joints

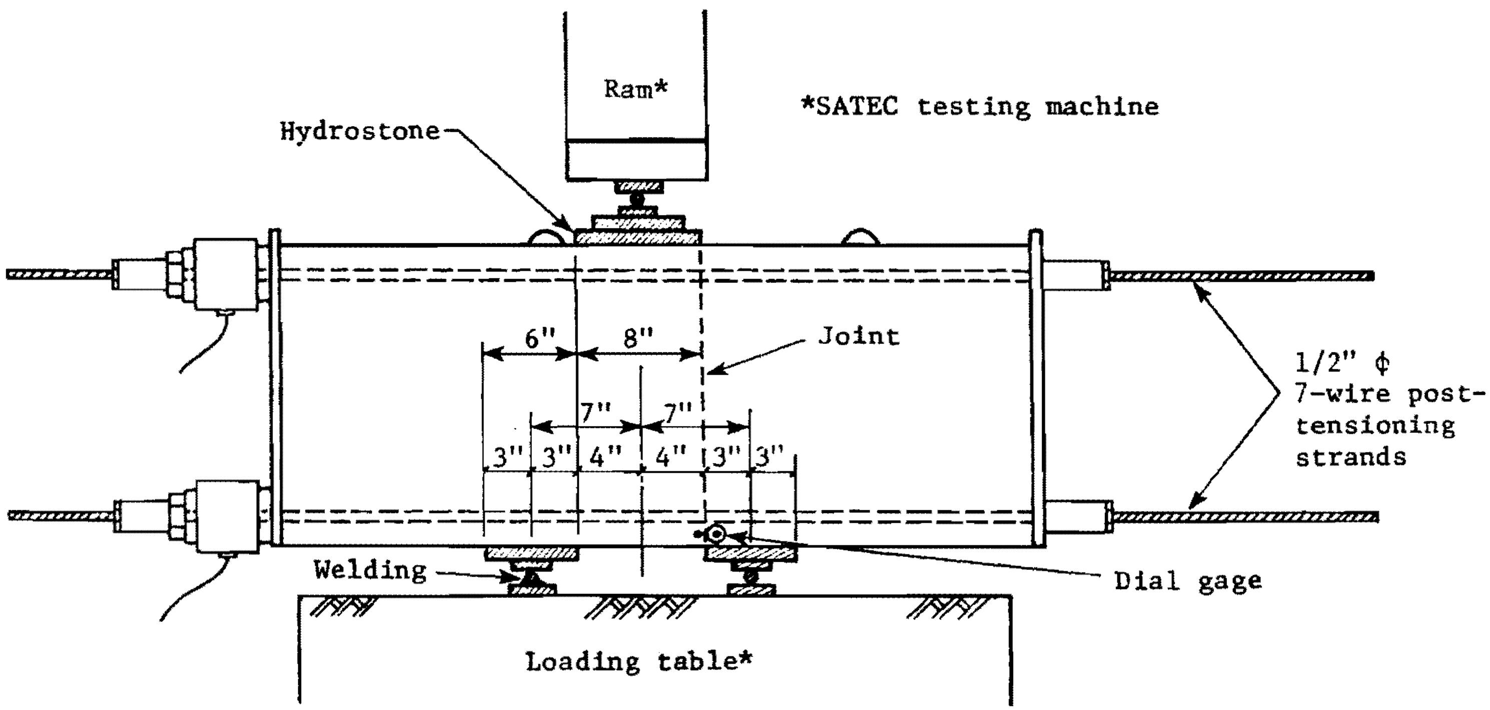
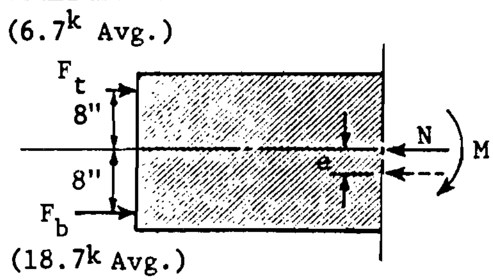


Fig. 3.2 Shear test arrangement

TABLE 3.1 STRESSES DUE TO PRESTRESSING

	Specimen	N or $F_t + F_b$ (kips)	M or $8(F_b - F_t)$ (k-in)	e or M/N (in)	Stresses* $-N/A \pm M/S$ (psi) Top	Bottom
W/O EPOXY	No-Key	25.3	92.0	3.64	38	-882
	Single Key	26.0	86.4	3.32	1	-865
	Multiple Keys	24.8	86.4	3.48	19	-845
	Average	25.4	88.3		19	-864
	(Standard deviation) (Coef. of Variance)	- (2%)	- (4%)		(19) -	(19) -
W/ EPOXY or SOLID	No-Key	25.7	101.6	3.95	80	-936
	Single Key	25.1	98.4	3.92	74	-910
	Multiple Keys	25.9	103.2	3.98	84	-948
	No-Joint 1	25.0	94.4	3.78	55	-889
	No-Joint 2	25.6	104.0	4.06	93	-947
	Average	25.5	100.3		77	-926
	(Standard Deviation) (Coef. of Variance)	- (2%)	- (4%)		(14) -	(26) -
Total Average		25.4	95.8	3.77	56	-903
(Standard Deviation) (Coef. of Variance)		(0.4) (2%)	(7.1) (7%)	(0.26) (7%)	(33) -	(39) -



* -: Compression
+: Tension

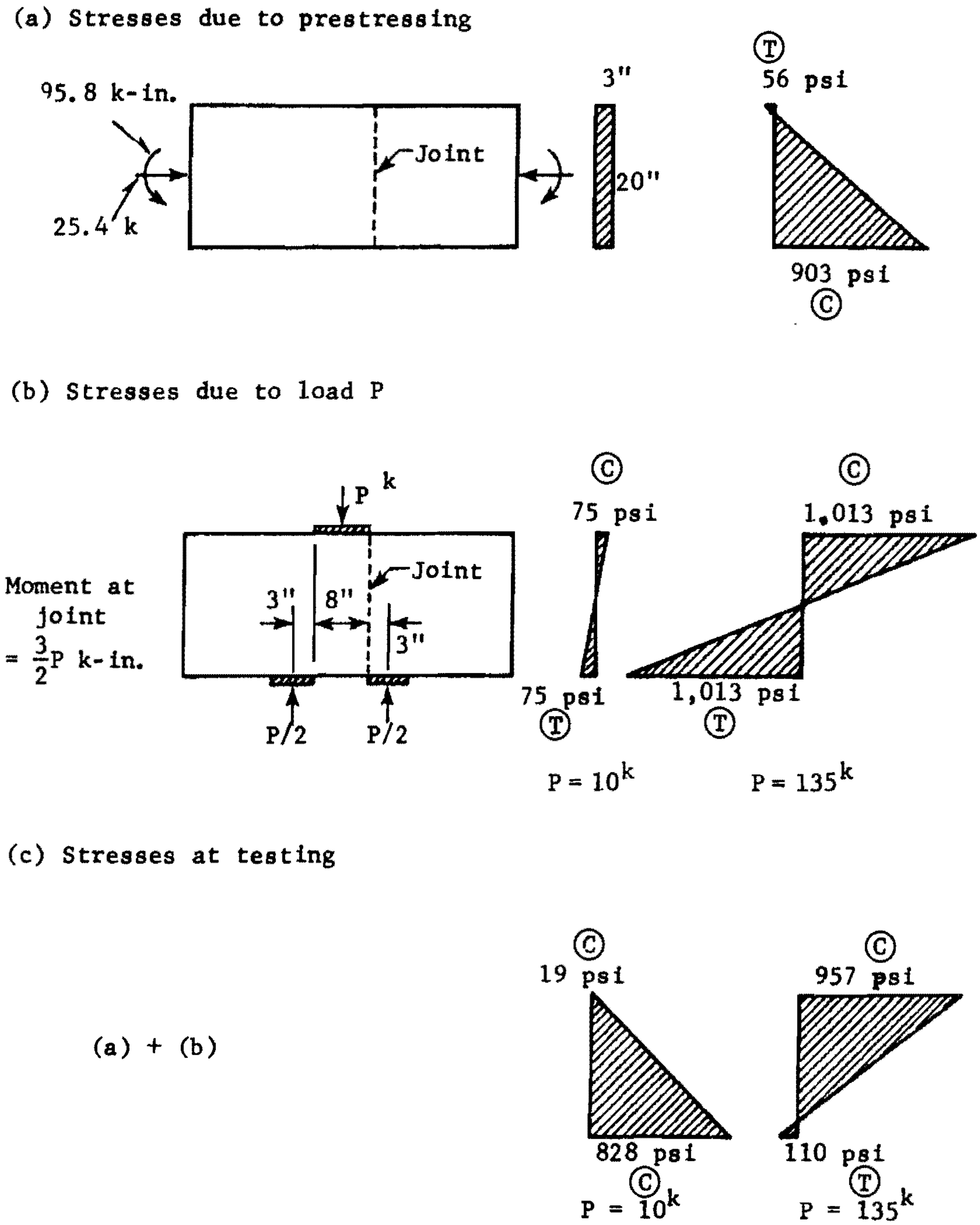


Fig. 3.3 Stresses at joint

throughout the test even for the highest level of load applied in any specimen (P = 135k).

NOTE: Since the specimen with the epoxied multiple key joint showed no damage in one entire half after the shear test, that undamaged portion was subjected to another shear test to obtain backup data for the monolithic specimen. This extra specimen is designated as No-Joint 2, while the original monolithic specimen is called No-Joint 1.

3.4.2 Load vs Slip. Figures 3.4 to 3.6 show the relationship between the applied load and the joint slip observed in the test.

(a) Specimens with nonepoxied joints - Figure 3.4 shows the data for the specimens with nonepoxied joints. As expected, each specimen slipped and failed at the joint. But, each type of joint configuration showed a definitely different load vs slip relationship.

The specimen with the no-key joint slipped at a load of 28 kips, which corresponds to a coefficient of friction of 0.55. The specimen, however, continued to carry load up to more than 70 kips with increasing slip at the joint. No damage was observed in the specimen. Joint surfaces were checked after the test and they seemed to be intact in appearance.

In the specimen with a single large key, the data from the dial gage indicated that slipping occurred from the beginning of the load application. However, it was felt that the slipping at such an early stage was unlikely and it is believed that something might be wrong with the displacement measuring system for this specimen. This subject will be discussed later in Chapter 4 in conjunction with the performance of joint configurations.

The crude data for the specimen with the single key joint indicate that (1) the inclination of the load vs slip curve is almost constant up to 34 kips, (2) from 34 to 84 kips, the slope is again constant but the value is less than that for the load range of 0-34 kips, and (3) from 84 kips to the major failure load, the slope of the curve is dramatically decreased showing slight resistance against slip. It is interesting to note that the slope of the load vs slip curve for the no-key joint takes

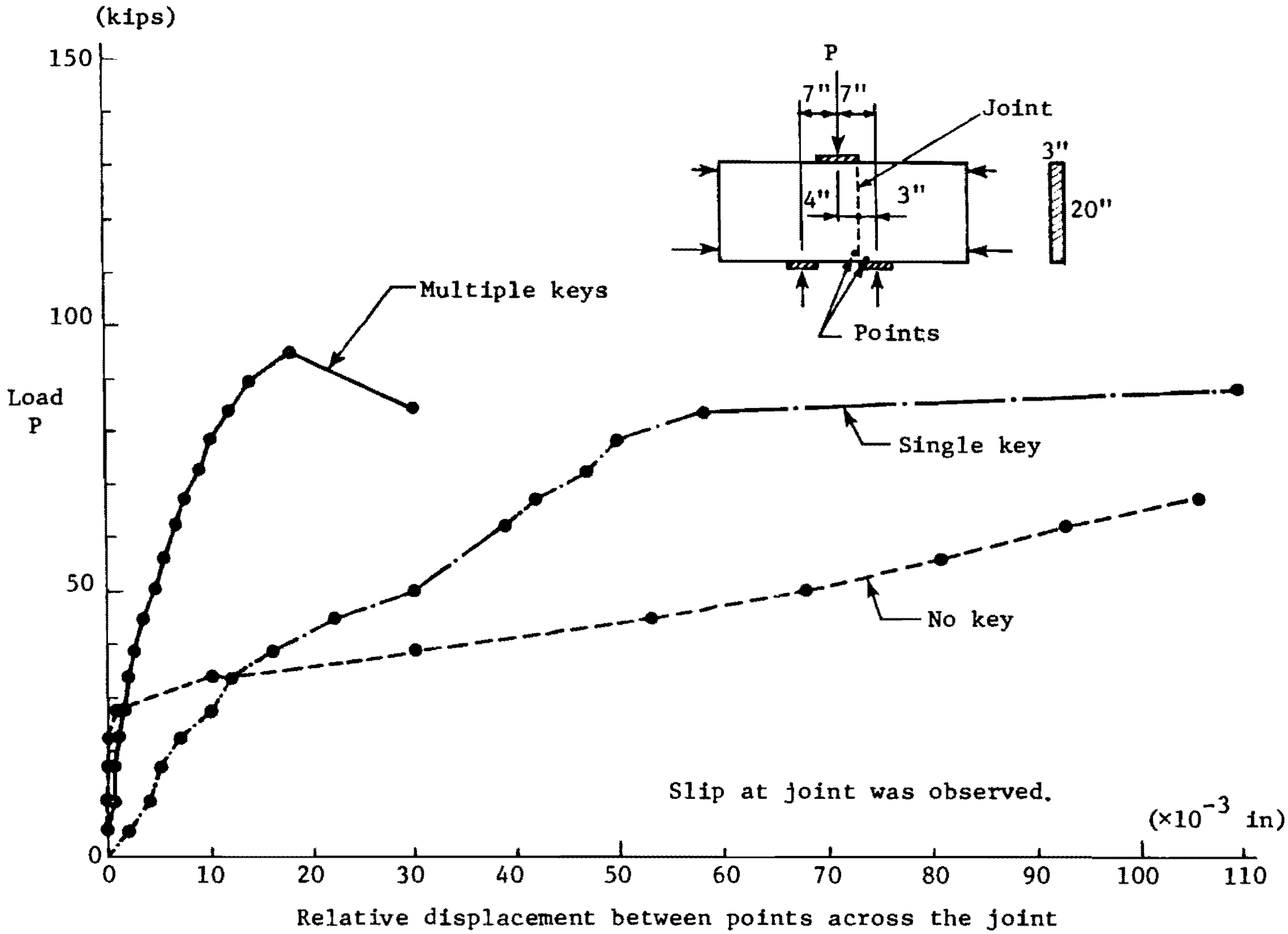


Fig. 3.4 Load vs "slip at joints" without epoxying

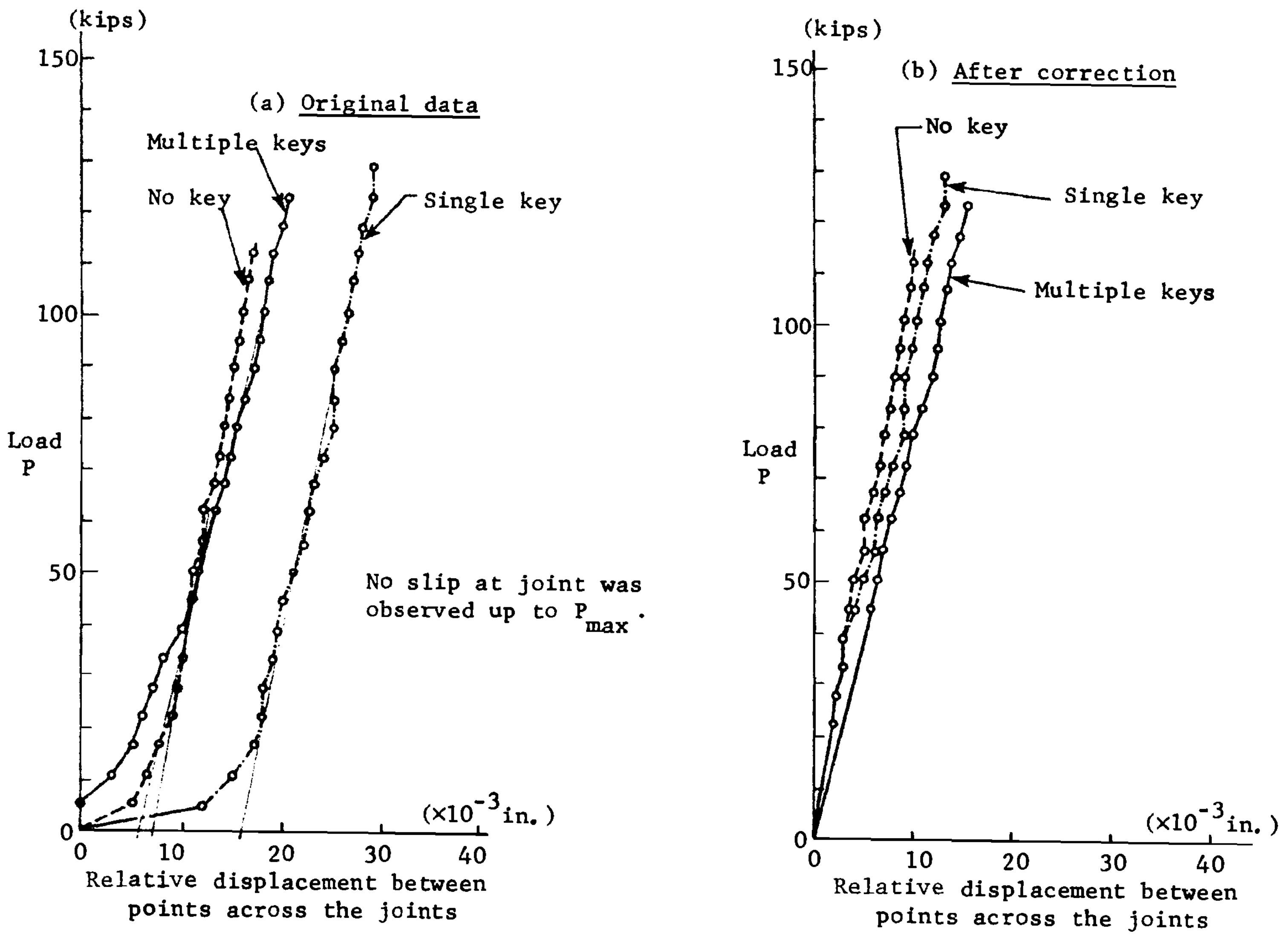


Fig. 3.5 Load vs "slip at joints" with epoxying

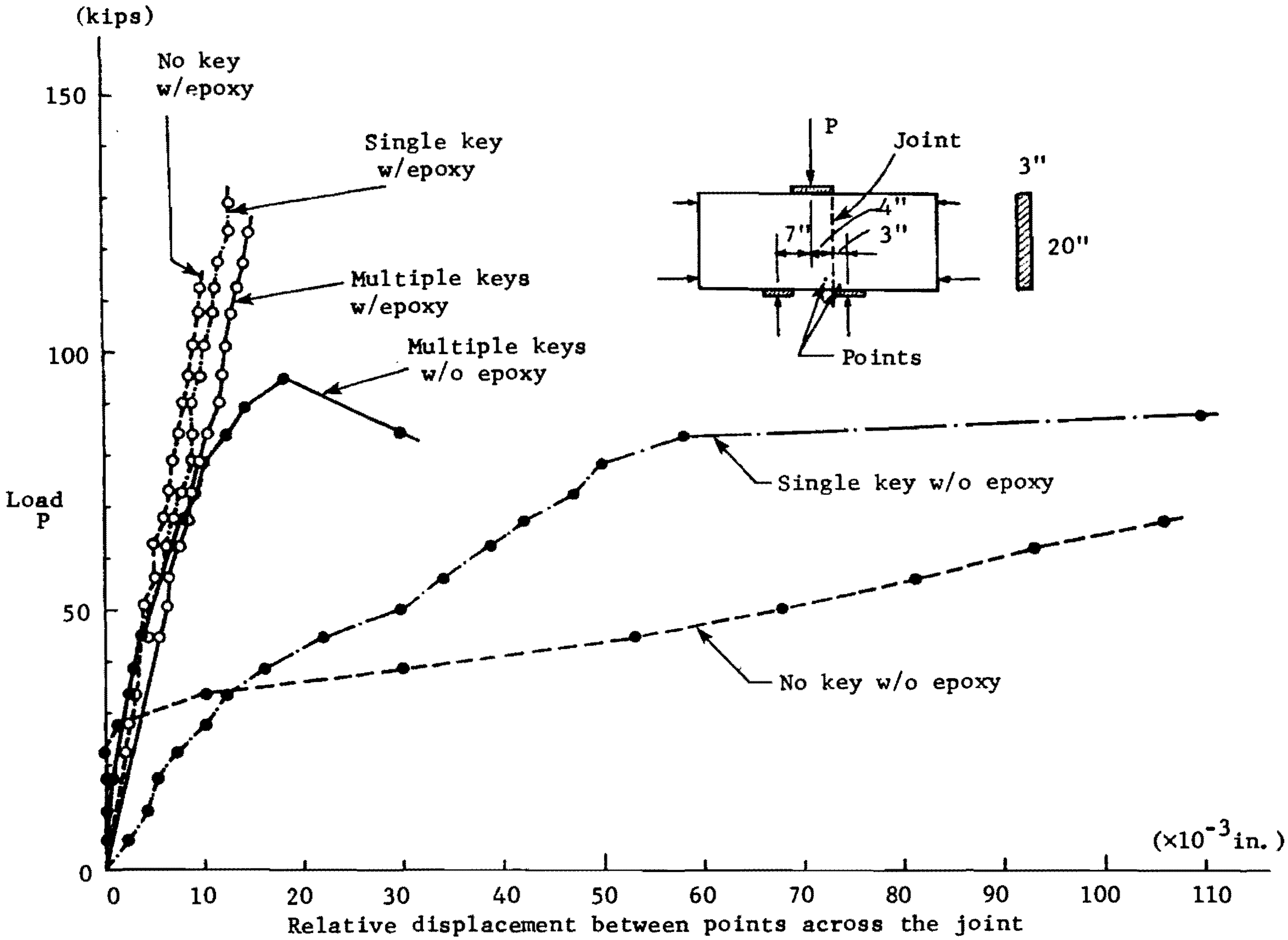


Fig. 3.6 Comparison of the results with and without epoxying

an intermediate value between those mentioned in stages (2) and (3). The major failure occurred in the single key portion.

The slip at the joint of the specimen with the multiple key joint occurred gradually as the load increased, showing substantially larger joint stiffness at higher load levels than the other nonepoxied joints. The slope of the load vs slip curve decreases continuously until the failure load is reached. This phenomenon is characteristic of the multiple key joint. Direct shear failure took place in the multiple keys.

The relative loads at given slips were substantially higher for the multiple key joint.

(b) Specimens with Epoxied Joints - The relationship of applied load and relative joint displacement for the specimens with epoxied joints is shown in Fig. 3.5. The figure includes two sets of load vs relative displacement curves: Fig. 3.5(a) shows plots with original data; and Fig. 3.5(b) shows plots with corrected data. In the test, no slip at the joints was observed visually up to the failure load. On the other hand, the uncorrected data of the relative displacement showed large slips at a very early loading stage. Since it was thought that these early stage slips were highly unlikely and represent instrument error, the data were corrected using overall slopes of the plotted curves. The corrected data plotted in Fig. 3.5(b) indicate no slips and agree with the visual observation. Again, it was concluded that something was wrong, for some unknown reasons, with the relative displacement measuring system, which worked well for the first two specimens; that is, the specimens with nonepoxied no-key and multiple key joints.

As seen in Fig. 3.5(b), all three specimens with epoxy joints gave almost the same load vs relative displacement curves.

The results of all jointed specimens are compared in Fig. 3.6. These will be discussed in Chapter 4.

3.4.3 Concrete Strength, Prestressing Force, and Maximum Load. Concrete strength of the control cylinders at the time of the applied shear test, prestressing forces just before the test and at the maximum loads, and the maximum applied loads are summarized in Table 3.2 and

TABLE 3.2 CONCRETE STRENGTH, PRESTRESSING FORCE AND MAXIMUM LOAD

Type of Joint	Without Epoxying: Concrete Strength f_c (psi)	Without Epoxying: Prestressing Force Before Test (kips)	Without Epoxying: Prestressing Force At P_{max} (kips)	Without Epoxying: Maximum Load P_{max} Test (kips)	Without Epoxying: Maximum Load P_{max} Predicted (kips)	With Epoxying: Concrete Strength f_c (psi)	With Epoxying: Prestressing Force Before Test (kips)	With Epoxying: Prestressing Force At P_{max} (kips)	With Epoxying: Maximum Load P_{max} Test (kips)	With Epoxying: Maximum Load P_{max} Predicted (kips)
No-Key	(A) 7,450 (B) 6,720	(F_t) 6.9 (F_b) 18.4 (F_p) 25.3	6.7 18.4 25.1	73 (28)*	- (20-35)*	(A) 6,510 (B) 6,200	(F_t) 6.5 (F_b) 19.2 (F_p) 25.7	6.5 19.4 25.9	116	109
Single Key	(A) 6,790 (B) 6,630	(F_t) 7.6 (F_b) 18.4 (F_p) 26.0	7.3 18.9 26.2	89 (34)*	40-60 (20-36)*	(A) 6,270 (B) 5,940	(F_t) 6.4 (F_b) 18.7 (F_p) 25.1	6.5 18.8 25.3	134	108
Multiple Keys	(A) 7,450 (B) 7,000	(F_t) 7.0 (F_b) 17.8 (F_p) 24.8	6.8 18.8 25.6	96 (60-80)*	44-68 (-)*	(A) 6,460 (B) 6,440	(F_t) 6.5 (F_b) 19.4 (F_p) 25.9	6.5 19.4 25.9	124	109

f_c = Average of 3 cylinders. *Slipping load.

	f_c (psi)	Prestressing (K) Before	Prestressing (K) At P_{max}	P_{max} (K) Test	P_{max} (K) Predicted
Solid (No-Joint)	(1) 6,230	(F_t) 6.6 (F_b) 18.4 (F_p) 25.0	6.6 18.6 25.2	118	108
	(2) 6,440	(F_t) 6.3 (F_b) 19.3 (F_p) 25.6	6.3 19.6 25.9	134	109

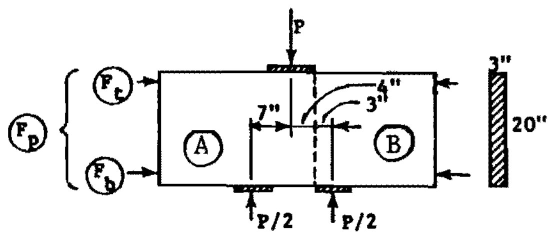


Fig. 3.7. The shear forces on the joints are one-half the applied loads. Table 3.2 also includes predicted maximum loads, for which details are presented in Chapter 4.

As shown in Fig. 3.7, overall average of the concrete strength at the ages of testing was 6550 psi and the coefficient of variance was 7%.

It was confirmed that the top and bottom prestressing strands picked up the moment due to the loading during the test, but the force changes in the tendons were very small. The average prestressing force was 25.4 kips and the centroidal concrete stress was 424 psi with the coefficient of variance of 1.7%.

Slip loads for the three specimens with the nonepoxied no-key, single key, and multiple key joints were 28 kips, 34 kips,* and 60-80 kips, respectively. In the same order, the maximum loads were 73 kips, 89 kips, and 96 kips for the three specimens with nonepoxied joints. All three specimens with epoxied joints, including the no-key joint, seemed to attain much higher failure loads and were very similar to those for a monolithic segment. The maximum loads for the epoxy joint and monolithic specimens ranged from 116 kips to 134 kips. Thus, a very impressive increase in failure load was realized by utilizing an epoxy bonding agent.

3.4.4 Crack Pattern at Failure

(a) Specimen with Nonepoxied No-key Joint - As mentioned before, no damage was observed.

(b) Specimen with Nonepoxied Single Key Joint - Figures 3.8 and 3.9 show the crack pattern of this specimen. Major cracking occurred at the top end of the male key. The cracks extended into the specimen with a downward angle of about 45°. The cracks were also observed along the shear plane of the male key. After the test, segments were separated. Large and wide cracks were observed in the planes of the key reinforcement. Some of the concrete was spalled off.

(c) Specimen with Nonepoxied Multiple Key Joint - Figure 3.10 shows the failure pattern of the specimen. Diagonal cracks were observed in each key. Generally speaking, cracks were localized in the neighborhood

---

*Obtained from the corrected load vs slip curve (see Chapter 4).

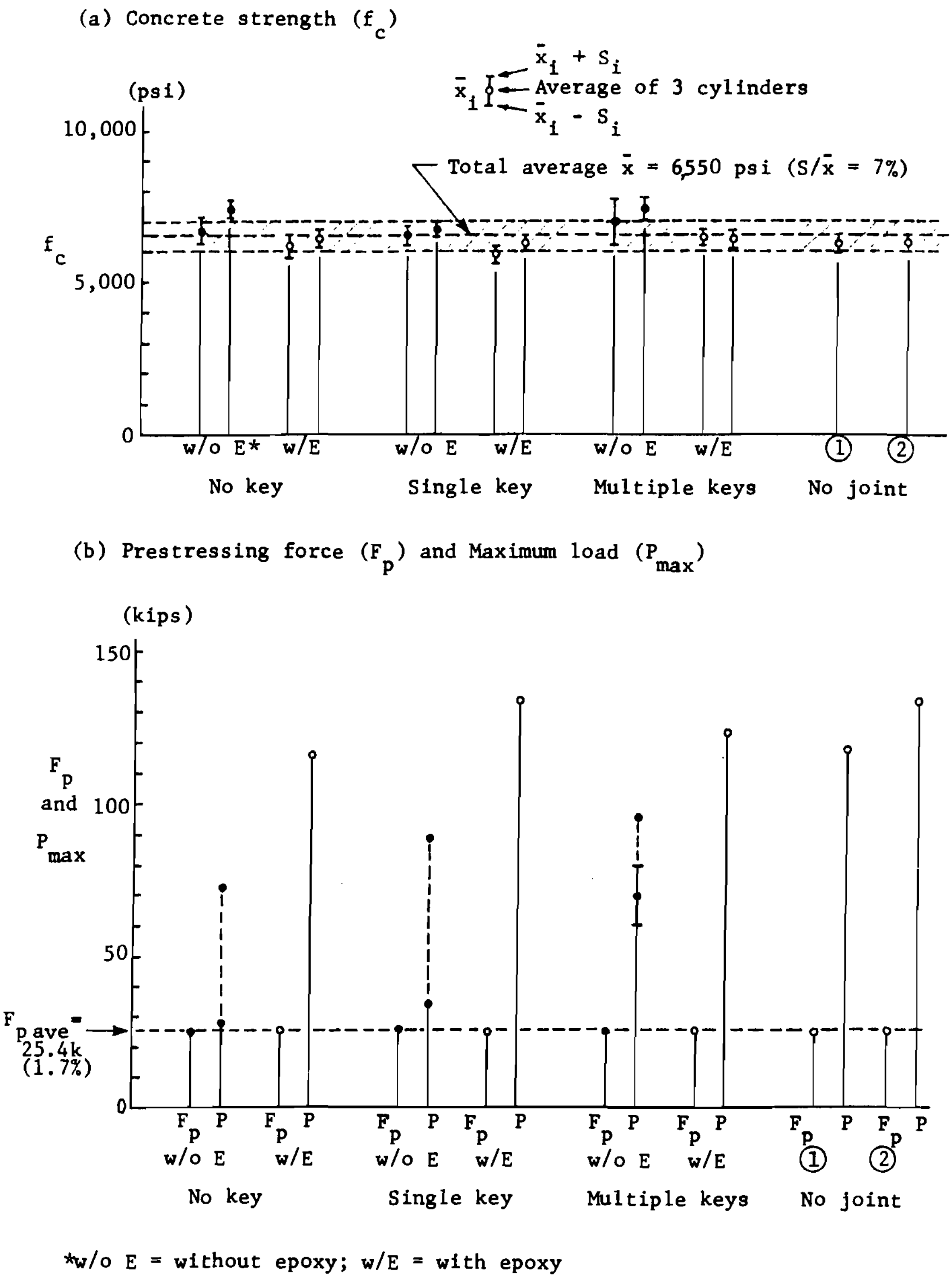


Fig. 3.7 Comparison of f_c, F_p and P_{max}

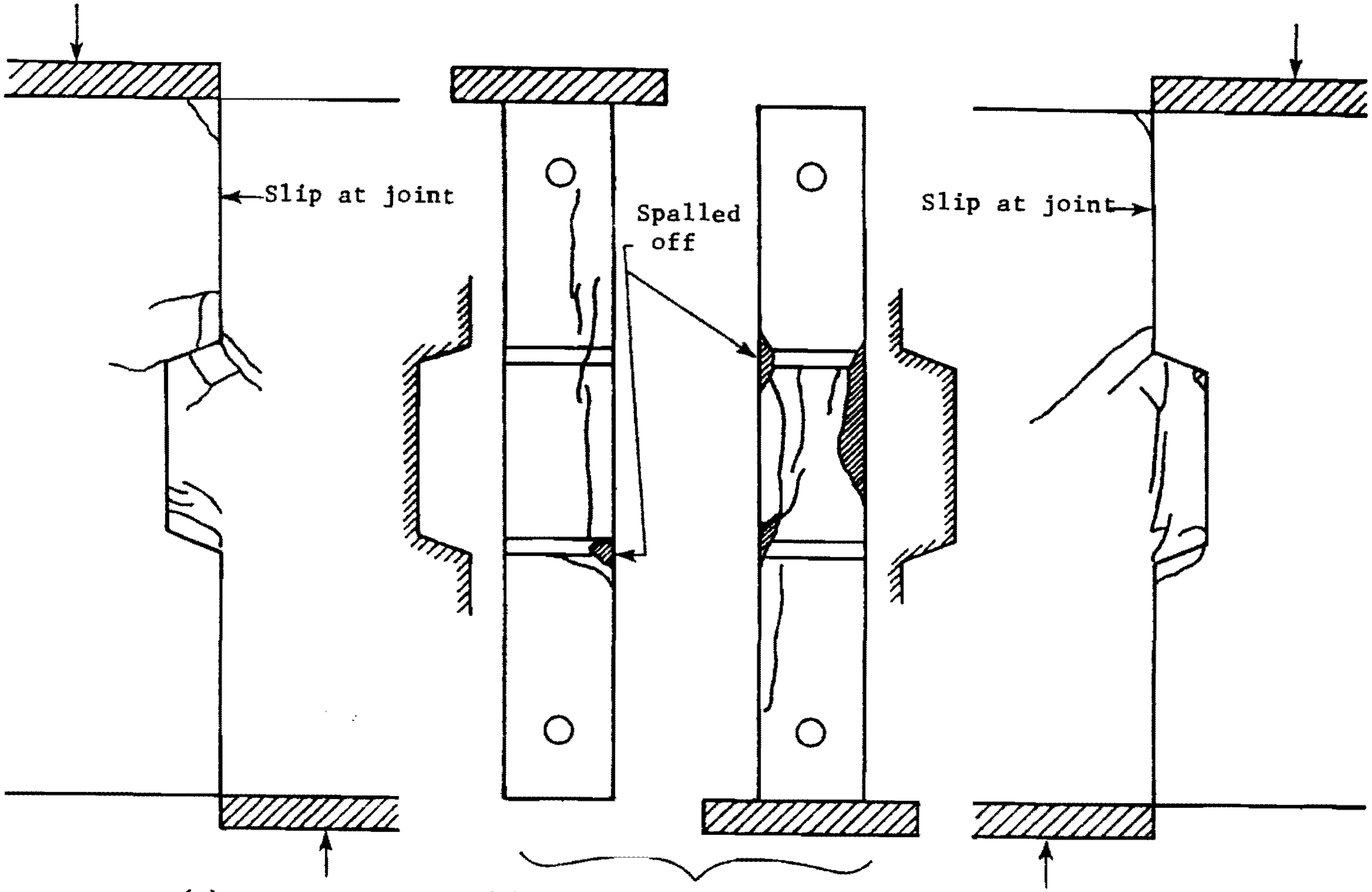


Fig. 3.8 Crack pattern at failure (single key joint w/o epoxying)

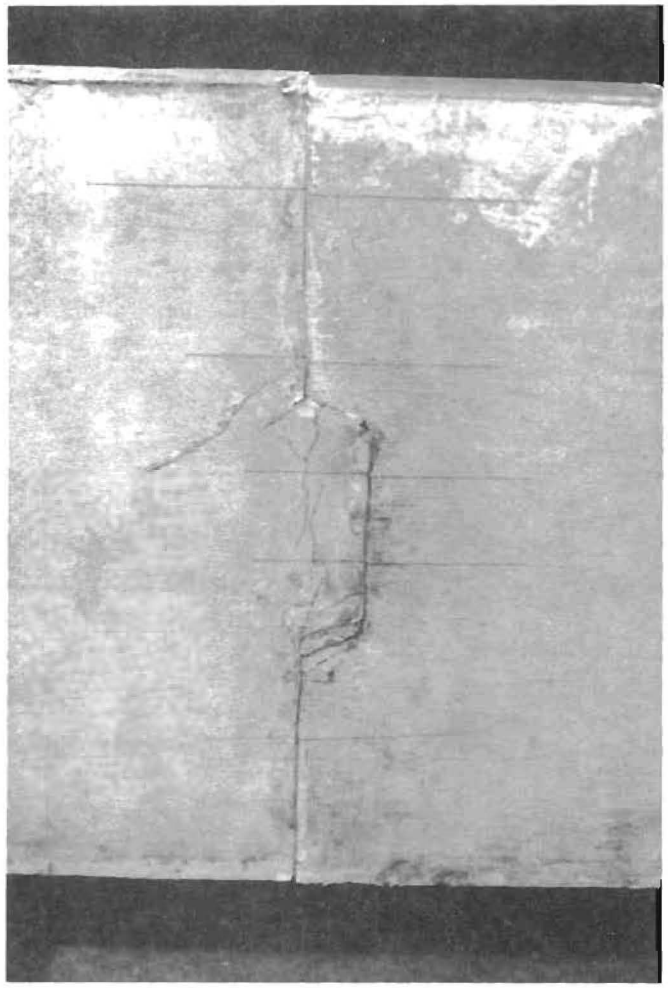
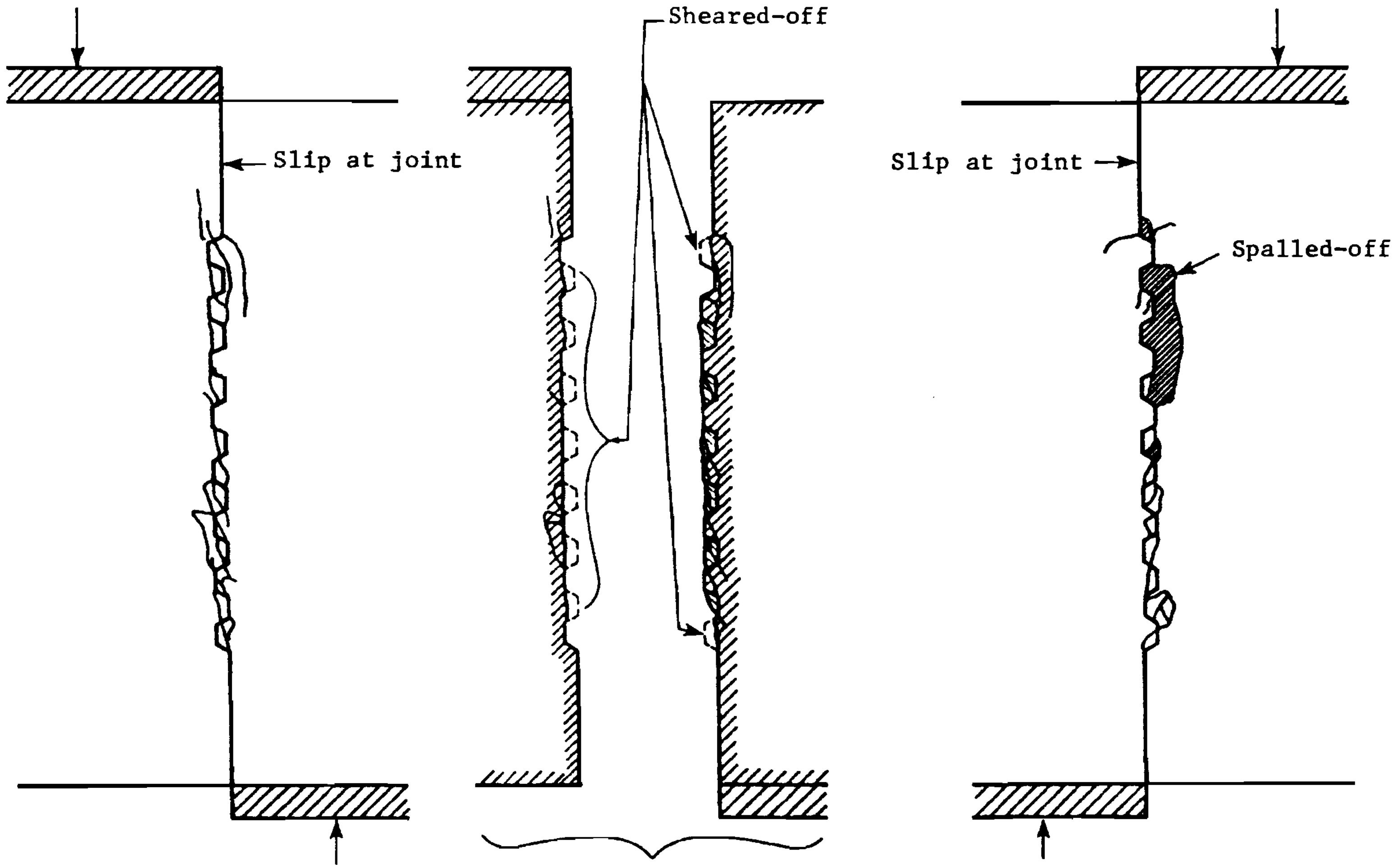


Fig. 3.9 Crack pattern at failure (single key joint w/o epoxying)



a) b) Joint after test c)

Fig. 3.10 Crack pattern at failure (multiple key joint w/o epoxying) of the keys. Keys on one side of the segments were completely sheared off, as shown in Fig. 3.11. The failure pattern was regarded as direct shear failure.

(d) Specimen with Epoxied No-key Joint - Figure 3.12(a) and Fig. 3.13 show the crack pattern of the specimen. The major shear crack runs down along the joint from the top to the midheight where the crack leaves the joint and goes toward a support. Bearing failure was observed underneath the loading plate. This failure pattern may be regarded as combination of shear and bearing failures. There was no slip at the joint.

(e) Specimen with Epoxied Single Key Joint - Figure 3.12(b) shows the crack pattern at failure for the specimen. Cracks appeared in the upper half of the specimen propagating from the bottom of the loading plate. This is considered to be a bearing failure. The joint was intact.

(f) Specimen with Epoxied Multiple Key Joint - Figure 3.12(c) shows the crack pattern at failure for the specimen. This is essentially the same as that of the specimen with the epoxied single key joint. Bearing failure occurred, and the joint was intact.

(g) Specimens with No Joints (Monolithic) - Figures 3.14 and 3.15 show the crack patterns of the monolithic specimens. As seen in Fig. 3.14, crack patterns of the two specimens were almost identical. The specimens had a combination failure of shear and bearing.

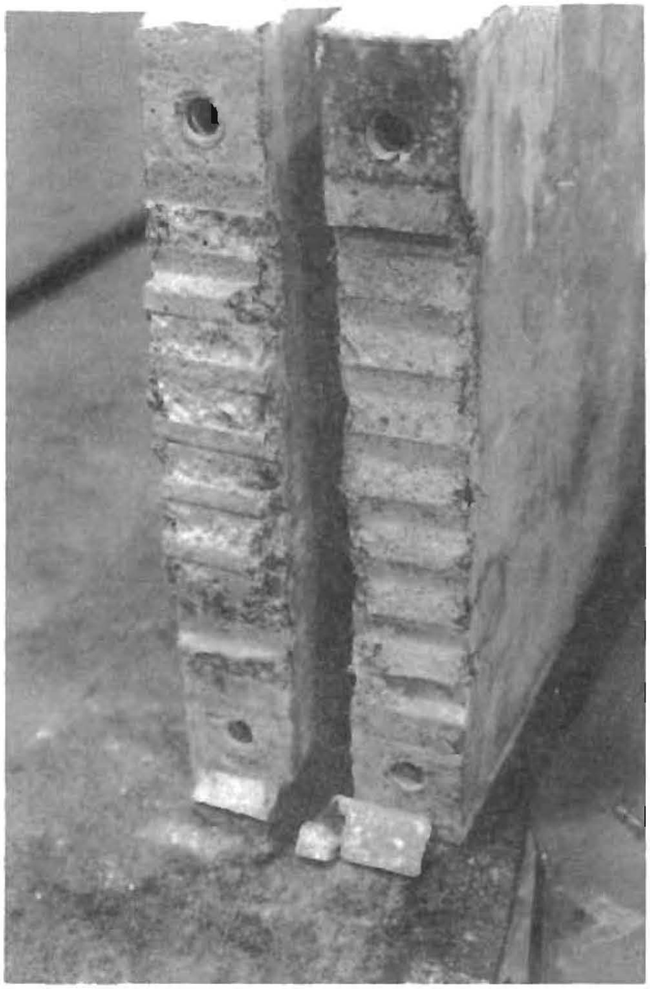


Fig. 3.11 Sheared-off joint after the test
(multiple key joint w/o epoxying)

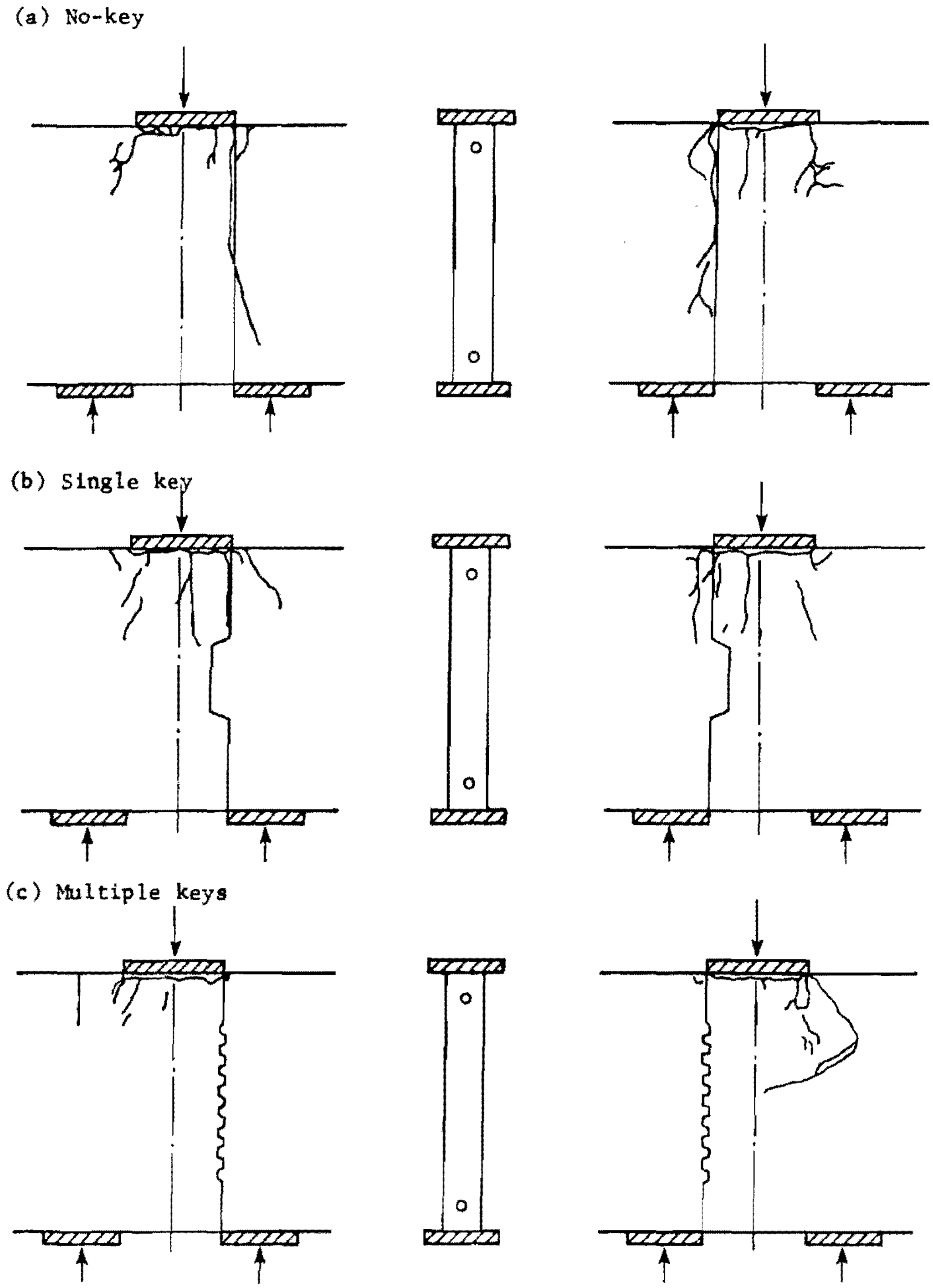


Fig. 3.12 Crack patterns at failure (specimens w/epoxying)

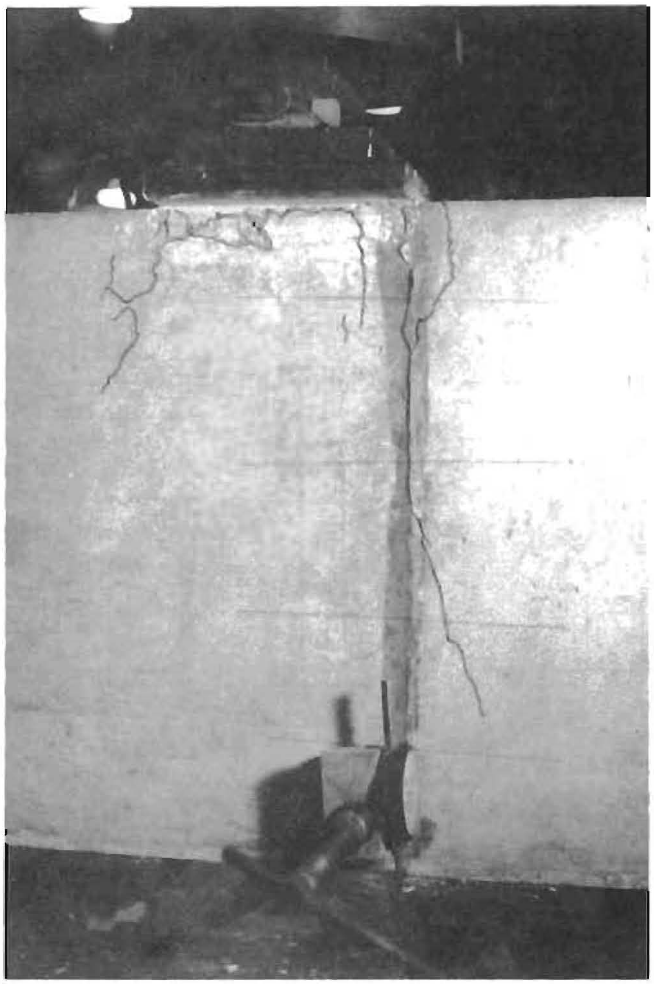
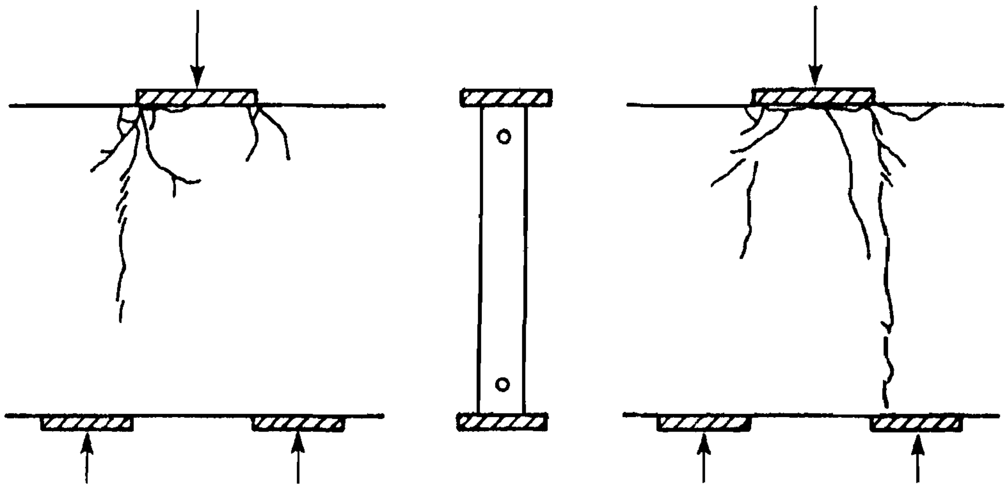


Fig. 3.13 Crack pattern at failure
(epoxied, no-key joint)

(a) 1



(b) 2

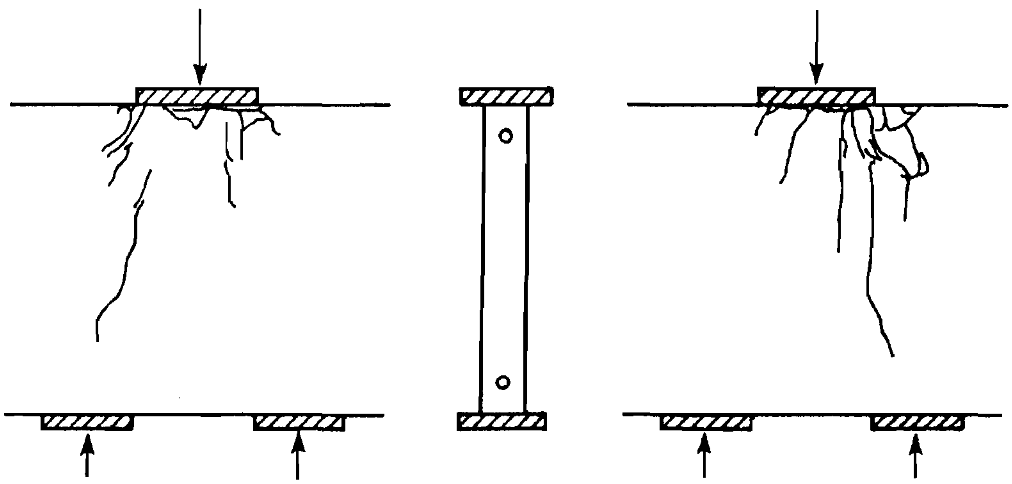


Fig. 3.14 Crack patterns at failure (no-joint)

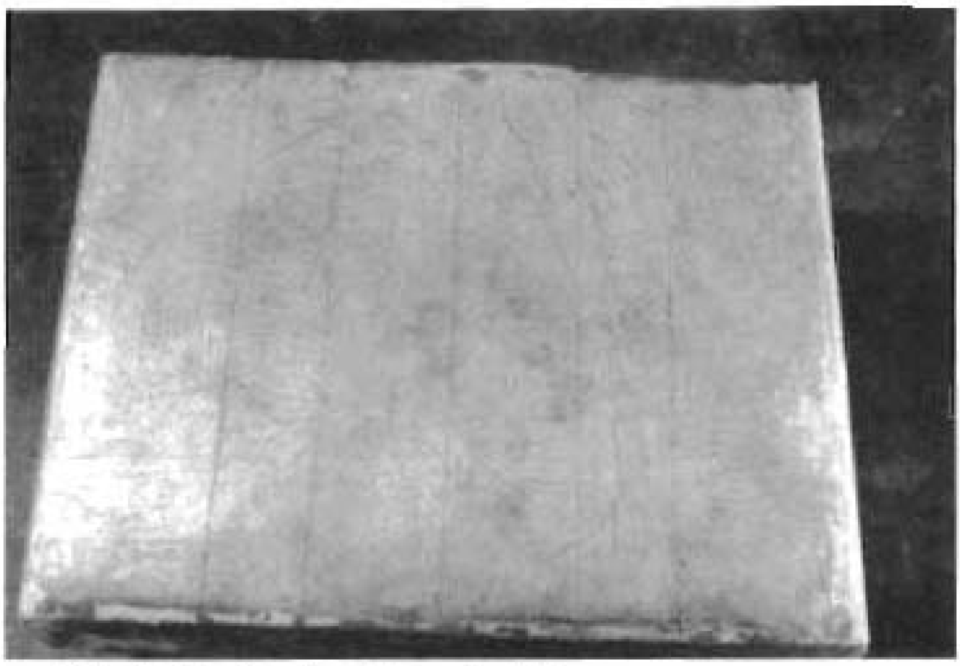


Fig. 3.15 Crack pattern at failure
(monolithic, no-joint)

CHAPTER 4

DISCUSSION OF THE RESULTS

4.1 Capacity of the Specimen

Flexural, shear, and bearing capacities of the test specimens, if assumed to be monolithic specimens, were calculated as described in this section. The calculated maximum loads P_{max} for each type of loading, which could be carried by the specimen, assuming monolithic action, are summarized in Table 4.1. In these calculations, the capacity reduction factor ϕ was taken as unity. In Table 4.1 the maximum test loads and the type of failure obtained in the tests are also shown for comparison.

4.1.1 Flexural Capacity. In accordance with ACI 318-77, Section 18.7.2, the flexural strength for each specimen, if monolithic, was determined using an effective ultimate prestressing force based on

$$f_{ps} = f_{se} + 10 + \frac{f'_c}{100\rho_p} \quad \text{(ksi)}$$

This equation should strictly only be applied to members with unbonded prestressing tendons and with $f_{se} \geq 0.5f_{pu}$. In the tests, f_{se} was only $0.45f_{pu}$ for the bottom tendons,but it was felt adequate to use this equation with measured prestressing forces and concrete strength. The limiting applied load P for a flexural failure is then obtained from the following formulas, assuming a uniformly distributed testing load application.

$$\rho_p = \frac{A_{ps}}{bd} = \frac{0.153}{3 \times 18} = 0.00283$$

$$T' = A_{ps}f_{ps}$$

$$a = \frac{T'}{0.85f'_c b}$$

$$M_n = T'(d - \frac{a}{2})$$

TABLE 4.1 CAPACITY OF THE TEST SPECIMENS IF MONOLITHIC

(In terms of maximum applied load, P_{max})

Type of Joint		Calculated Values Assuming Monolithic Action Flexure (kips)	Shear (kips)	Bearing (kips)	Maximum* Test Load (kips)	Nature of Failure
Without Epoxy	No key	166	111	152	78 (28)	Slip along Joint
	Single key	163	110	139	89 (34)	Shear Failure in Key
	Multiple keys	162	110	152	96 (60 ~ 80)	Shear Failure in Keys
With Epoxy	No key	167	109	133	116	Combined Shear and Bearing Failure
	Single key	163	108	128	134	Bearing Failure
	Multiple keys	169	109	132	124	Bearing Failure
Monolithic (No Joint)	1	156	108	127	118	Combined Shear and Bearing Failure
	2	168	109	131	134	Combined Shear and Bearing Failure

*, **Values in parentheses show slipping loads.

$$M_{max} = \frac{5}{2} P$$

$$\therefore P = \frac{2}{5} M_n$$

As seen in Table 4.1, the flexural capacity of the specimens was designed to be relatively high as compared to the shear capacity so that a premature flexural failure would not mask the joint shear capacity. No flexural failures occurred in the test series.

4.1.2 Section Shear Capacity. The calculated shear strength of each test specimen, assuming monolithic action and ignoring the joint effect, was calculated following ACI 318-77, Sections 11.4 and 11.5, which were adopted by AASHTO as Article 1.6.13 in the 1980 Interim changes.

-Shear strength provided by concrete V_c

$$\text{(flexural shear)} \qquad V_{ci} = 0.6\sqrt{f'_c}\, b_w d + V_d + \frac{V_i}{M_{max}} M_{cr}$$

$$\text{(web shear)} \qquad V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc}) b_w d + V_p$$

Since the shear span of the specimens is very short, web shear governs. Therefore, $V_c = V_{cw}$. The actual shear capacity should be somewhat higher than the V_{cw} estimate due to the short shear span.

-Shear Strength provided by shear reinforcement V_s

$$V_s = A_v f_y \frac{d}{s}$$

The nominal shear strength $V_n = (V_c + V_s)$ was calculated using measured f'_c and f_{pc} for each specimen. Other values are given below.

$b_w = 3$ in.

$d = 18$ in.

$V_p = 0$

$$A_v = 2 \times \frac{\pi}{4} \times \left(\frac{6}{25.4}\right)^2 = 0.0877 \text{ in}^2$$

$f_y = 61.1$ ksi

$s = 3$ in.

$P = 2 \times V_n$

This calculated monolithic shear strength was not developed in any of the jointed test specimens without epoxy but was fully developed in all monolithically cast and all epoxy jointed test specimens.

4.1.3 Bearing Capacity. The calculated bearing strength of each test specimen, assuming monolithic action, was determined following ACI 318-77, Section 10.16, and AASHTO Article 1.5.36, using the following equations:

$$P = 0.85 f'_c A_1$$

$$A_1 = 3 \times 8 = 24 \text{ in.}^2$$

The measured f'_c at the time of the shear test was used in the calculation for each specimen.

As seen in Table 4.1, the bearing capacity of the specimens was designed to be higher than the shear capacity. The two epoxy jointed specimens with keys which underwent bearing-type failures both failed within 6% of the predicted bearing capacity.

4.2 Shear Strength of the Joints

The method for determining the shear strength of joints in precast segmental construction has not been standardized. There are a wide range of approaches which have been suggested. The shear strength of each specimen has been computed by various applicable theories as detailed in this section. The results of these calculations are summarized in Table 4.2. For comparison, both the calculated shear strength assuming monolithic action, and the measured shear strength, and the major slipping loads are also given.

4.2.1 Shear Friction. The shear strength of the nonepoxied joint without keys was calculated using shear friction concepts. Values of the coefficient of friction given by codes or obtained from test for concrete-to-concrete smooth interfaces are as follows:

	μ
ACI 318-77 [13] and AASHTO [17]	0.7-1.0
PCI Design Handbook [26]	0.4
Mast [12]	0.7-1.0
Franz [18]	0.7
Jones [19]	0.4
Gaston and Kriz [20]	$0.78 + \frac{43A_i}{N} = 0.88$

In many cases such as ACI 318-77 and AASHTO the case of rejoining match-cast units is not directly covered. Values of $\mu = 1.0$ are suggested for concrete placed against hardened concrete, while values of $\mu = 0.7$ are used for concrete placed against as-rolled structural steel. This case may be in between.

In the shear friction method of calculation in reinforced concrete design, it is assumed that all the shear resistance is due to friction across the crack faces. The ACI Building Code and AASHTO Specifications, therefore, use artificially high values of the coefficient of friction in order to compensate for the neglect of dowel action of the reinforcement crossing the crack and resistance to the shearing off of protrusions on the crack faces.

For precast construction, it has been reported [16,25,28] that an externally applied compressive stress acting transversely to the shear plane is additive to the reinforcement parameter ρf_y in calculations of the shear transfer strength of both initially cracked and uncracked concrete.

It has also been reported [18,19,25] that the coefficient of friction and the shear transfer strength are not significantly affected by the presence of moment in the crack or joint, providing the applied moment is less than or equal to the flexural capacity of the section.

The shear strength of the joints for the test specimens with nonepoxied joints was computed in accordance with the ACI Building Code [13], the PCI Design Handbook [26], and an alternate shear transfer design method proposed by Mattock [25]. The results are shown below. In the calculation, $\phi = 1.0$ was used, since the material strength and specimen dimensions were accurately known.

(a) ACI 318-77, Section 11.7

$V_n = \mu A_{vf} f_y$, $V_n = \mu N$, $P_{max} = 2\ V_n$

Dry Joint	V_n (kips)		
	(μ = 0.4)	(μ = 0.7)	(μ = 1.0)
No-key	10.1	17.7	25.3
Single key	10.4	18.2	26.0
Multiple keys	9.9	17.4	24.8

As can be seen from Table 4.2, the values based on either μ = 0.7 or 1.0 for concrete placed against hardened concrete are conservative predictors of the maximum load values but underestimate the value at which significant slip occurred.

(b) PCI Handbook, Section 5.6

In the application to precast concrete connections, the use of an "effective shear friction coefficient," μ_e is recommended.

$$V_n = A_{vf} f_y \mu_e \qquad V_n = N\mu_e$$

$$\mu_e = \frac{1000\,A_{cr}\,\mu}{V_n} \qquad \therefore V_n = \frac{N\cdot 1000\,A_{cr}\,\mu}{V_n}$$

$$\therefore V_n = \sqrt{1000\,A_{cr}\,N\,\mu}$$

where A_{cr} = area of contact surface.

For concrete-to-concrete connections with a smooth interface surface, PCI recommends that a value of 0.4 be used for μ. This gives the following values for V_n.

Dry Joint	V_n (kips)
	(μ = 0.4)
No-key	24.6
Single key	25.0
Multiple keys	24.4

TABLE 4.2 SHEAR CAPACITY OF THE TEST SPECIMENS
(In terms of maximum applied load, P_{max})

Type of Joint		ACI 318 & AASHTO (if monolithic)	Calculated Values for Shear Strength of Joint (kips) (In terms of maximum applied load, $P_{max} = 2 V_n$)									Max.* Test Load (kips)	Nature of Failure
			Shear Friction ACI 318-77 Sec. 11.7			Shear Friction PCI Handbook	Mod. Shear Friction Mthd Ref. 25	ACI Corbel Theory using Shear Friction	ACI Corbel Theory Sec. 11.9	PCI Handbook Corbel Sec. 5.11	Nom. Shear Key $V_n = vbh$ $v = 6\sqrt{f'_c}$ / $8\sqrt{f'_c}$		
			$\mu = 0.7$	$\mu = 1.0$	$\mu = 1.4$								
Without Epoxy	No key	111	35	51		49	84					78 (28)	Slip Along Joint
Without Epoxy	Single key	110	36	52		50	85	50	55	35	44 48	89 (34)	Shear Failure in Joint
Without Epoxy	Multiple keys	110	35	50		49	83				48 57	96 (60-80)	Shear Failure in Joint
With Epoxy	No key	109			72							116	Combined Shear & Bearing Failure
With Epoxy	Single key	108			70							134	Bearing Failure
With Epoxy	Multiple keys	109			73							124	Bearing Failure
No Joint	1	108			70							118	Combined Shear & Bearing Failure
No Joint	2	109			72							134	Combined Shear & Bearing Failure

* Values in parentheses show slipping loads.

These values are very close to the ACI-AASHTO values based on $\mu = 1.0$ in the ACI procedures and are again conservative predictors of ultimate load, but are significantly greater than the load at which slip began.

(c) Modified Shear Friction Method of Mattock [25]

$$v_u = 400 \text{ psi} + 0.8\rho f_y$$

$$V_n = 400\, b_w d + 0.8N$$

Dry Joint	V_n (kips)
No-key	41.8
Single key	42.4
Multiple keys	41.4

This method overestimates the initial slip load of the keyless and the single key specimens and also overestimates the ultimate load on the keyless specimen.

As described in Sec. 3.4.2 and shown in Fig. 3.4, the jointed specimen with no epoxy and no keys continued to carry load after the initial slip at the joint and reached a capacity of 73 kips. The load vs slip relationship was bilinear. It is supposed that the increased load was carried by resistance to the shearing off of the fine protrusions on the "flat" joint surfaces, and by dowel action of the unbonded prestressing tendons. It is unlikely that any design credence should be given to the increased load capacity after marked slip.

4.2.2 Single Key Joint

4.2.2.1 Behavior of the test specimen with a nonepoxied single key joint. The crack pattern of this specimen (Figs. 3.8 and 3.9) showed three types of major cracking characteristics: (1) flexural cracks starting at the junction of the top face of the male single key and the end face of the segment, (2) shear cracks in the shear plane of the male key, and (3) splitting cracks in the shear key reinforcement planes of both adjacent segments which were accompanied by spalling-off in the male key region. The flexural cracks (1) were first observed just before the major shear failure (2).

As described in Sec. 3.4.2 and shown in Fig. 3.4, the load vs slip curve for the specimen with the nonepoxied single key joint implied that slip occurred at the joint from the very beginning of the load application. A slip of 0.013 in. was recorded at P = 34 kips. Slip at the joint was also being observed visually by watching the relative displacement of straight lines drawn on the sides of the specimen across the joint. No visual slip was observed at the load level of 34 kips for the specimen. The load vs slip relationship was essentially trilinear.

Considering the above-mentioned facts and the test results for the specimen with the dry no-key joint, it was concluded that some type of seating error occurred in the slip gage and the load vs slip data were corrected, as shown in Fig. 4.1. The corrections were made so that the first linear portion of the original trilinear curve for the dry single key joint would agree with the initial slope of the load vs slip curves for the other specimens. The curve was shifted in proportion to the magnitude of the applied load. Since shear forces are first transferred by the friction in the contact surfaces with prestressing forces providing active resistance, it was considered reasonable that the initial portions of the load vs slip curves be assumed identical to each other.

Figure 4.1 indicates that considerable slip must take place in order for a single large key to act and to develop high shearing stresses. In other words, contributions of the friction and the key to initial slip loads are not additive. The corrected apparent slip load for the specimen with dry single key joint was 34 kips ($\mu = 0.61$), while that for the dry no-key joint was 28 kips ($\mu = 0.55$) as mentioned in the previous chapter.

Figure 3.6 shows that the ultimate shear transfer strength of the dry joint was somewhat improved by the existence of a key (from 73 to 89 kips). Even after the flexural failure of the key, the joint continued to carry the shearing load, but the slope of the load vs slip curve was much flatter than in the case of the dry no-key joint. It is supposed that the reinforcement in the vicinity of the key helped to maintain the load-carrying capacity. After the shear failure of the male key, which might have been accompanied by the splitting cracks in the key, the load dropped rapidly.

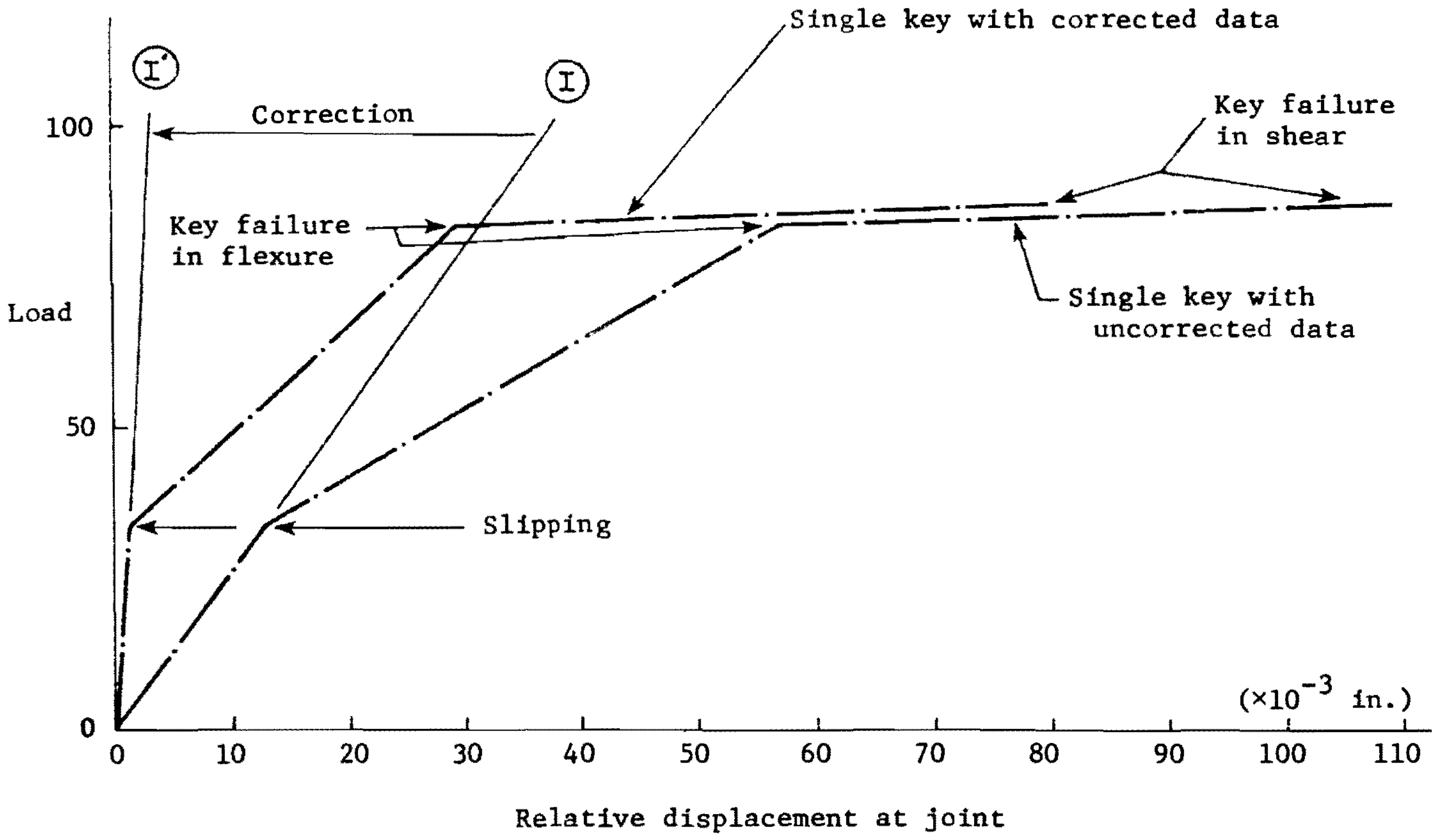


Fig. 4.1 Correction of load vs slip curve of single key joint w/o epoxying

4.2.2.2 Forces acting on single key. Figure 4.2 illustrates forces acting on a key of the specimen with the nonepoxied single key joint.

In Fig. 4.2(a), the probable forces on the single key just before slipping are shown. The resultant force on the key was obtained assuming uniform distribution of normal compressive stresses due to prestressing and no contact between top or bottom faces of the male and female keys, resulting in no forces acting on the top or bottom face of the male key. Maximum shear friction stresses before slipping was approximately 280 psi, which corresponds to a coefficient of friction of 0.61.

The probable forces on the key just before key failure in flexure are shown in Fig. 4.2(b). The resultant force was calculated using prestressing force, shear friction force and forces acting on the top face of the key. The shear friction force and forces acting on the top face of the key were determined using the load vs slip curves for the no-key and single key joints. It is assumed that the shear friction force would be the same as that of the no-key joint at an identical amount of slip. Uniform distribution of the force acting on the top face of the key is also assumed. The stresses in the key were calculated as follows:

-Average shear stress in the key base just before failure

$$v_{ave} = \frac{Q}{A} = \frac{(18 + 5.9)}{3 \times 6} \times 10^3 = 1{,}330 \text{ psi}$$

This corresponds to 0.20 f'_c or $16\sqrt{f'_c}$ ($> 8\sqrt{f'_c}$).

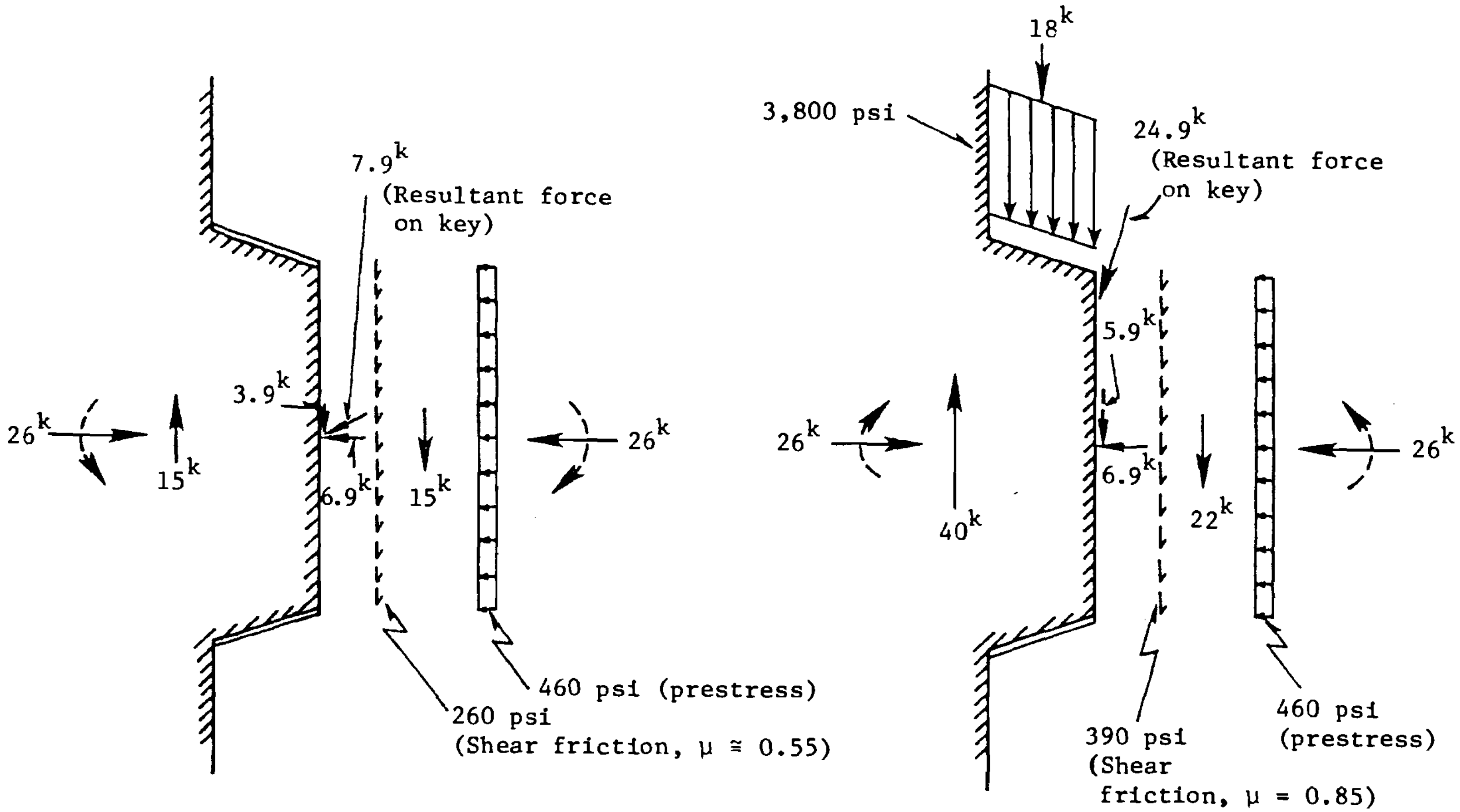
-Flexural tensile stress at the top reentrant corner

$$M = 18 \times \frac{3}{4} + 5.9 \times \frac{3}{2} = 22.4 \text{ k-in}$$

$$f_t = \frac{M}{S} - \frac{N}{A} = \frac{22.4 \times 10^3}{18} - 460$$

$$= 780 \text{ psi}$$

This corresponds to 0.12f'_c or $9.6\sqrt{f'_c}$ ($> 7.5\sqrt{f'_c}$).



a) Just before slipping (P = 30 kips).

b) Just before key failure (P = 80 kips)

Fig. 4.2 Forces acting on single key (without epoxy)

From this simple calculation, the relative magnitude of the shear force contribution by various components can be visualized. Maximum bearing stress on the key faces was approximately $0.6f'_c$ ($<0.85f'_c$).

4.2.2.3 Corbel analogy. Single key joints might be considered to be analogous to corbels. Hence, the shear strength of the dry single key joint was calculated using the design methods for corbels.

The ACI Building Code permits the use of the shear friction provisions for the design of corbels in which the shear span-to-depth ratio a/d is one-half or less, providing limitations on the quantity and spacing of reinforcement in corbels. ACI 318-77, Section 11.9, governs the design of corbels with a shear span-to-depth ratio a/d of unity or less. Provisions of Section 5.11 of the PCI Design Handbook would also apply to corbels. Shear strength of the dry single key joint was calculated using those methods as shown below. Results of a direct shear strength calculation will also be presented. In shear friction calculations a value of $\mu = 1.4$ was used on those parts of the shear friction plane where the concrete is monolithic but a value of $\mu = 0.7$ was used where match cast surfaces joined. It was felt that this surface condition is closer to that of concrete to steel than concrete placed against concrete and left undisturbed.

a) ACI 318-77, shear friction provisions

Key reinforcement $$A_{vf}f_y = 2 \times \frac{\pi}{4}(0.135)^2 \times 33.6$$

$$= 0.96 \text{ k}$$

Key portion $$V_{n_1} = (A_{vf}f_y + N)\mu$$

$$= (0.96 + 26.0 \times \frac{6}{20}) \times 1.4$$

$$= 12.3 \text{ k}$$

The rest of the joint $$V_{n_2} = 26.0 \times \frac{14}{20} \times 0.7$$

$$= 12.7 \text{ k}$$

Total shear strength $V_n = V_{n_1} + V_{n_2} = 25$ k

Therefore, $P = 2V_n = 50$ k.

b) ACI 318-77, corbel provisions

$$V_{n_1} = 6.5(1 - 0.5\,\frac{a}{d})(1 + 64\rho_v)\sqrt{f'_c}\;b_w d$$

$$= 6.5(1 - 0.5 \times \frac{0.75}{5.5})(1 + 64 \times 0.0017)\sqrt{6630} \times 3 \times \frac{5.5}{1000}$$

$$= 9.0 \text{ k}$$

$$V_{n_2} = N\mu = 26.0 \times 0.7 = 18.2 \text{ k}$$

$$V_n = V_{n_1} + V_{n_2} = 27.2 \text{ k}$$

$$P = 54.8 \text{ k}$$

c) PCI Design Handbook, corbel provisions

$$\left.\begin{array}{l} (1)\ A_s + A_n = \dfrac{1}{\phi f_y}\,[V_u(a/d) - N_u(h/d)] \\[2ex] (2)\ A_s + A_n = \dfrac{1}{\phi f_y}\left[\dfrac{2V_u}{3\mu_e} - N_u\right] \end{array}\right\}\ \text{Use the greater } A_s + A_n.$$

(Note that the sign of the N_u term has been changed to reflect a compressive force.)

After modification,

$$(1')\ V_n = \frac{A_{vf} f_y d + N(h/2)}{a}$$

$$= \frac{0.96 \times 5.5 + 26 \times \frac{6}{20} \times \frac{6}{2}}{0.75} = 38.2 \text{ k}$$

$$(2') \quad V_n = \sqrt{\frac{3}{2} \times 1000 b_w d\mu \, (A_{vf} f_y + N)}$$

$$= \sqrt{\frac{3}{2} \times 3 \times 5.5 \times 1.4(0.96 + 26.0 \times \frac{6}{20})} = 17.4 \text{ k}$$

The smaller V_n governs. Therefore, $V_n = 17.4$ k, $P = 34.8$ k.

Since the effect of the normal force is already included in the equations, no shear friction contribution will be included.

d) Shear keys with assumed shear distribution

For shear strength in the shear plane of the shear keys, Ferguson [34] writes as follows:

> The distribution of shear force on the key section is uncertain. If it is taken as parabolic, as for a homogeneous rectangular beam, the equation is
>
> $$V_n \geq (vbh)2/3 \ .$$
>
> The allowable shear in such a case is also not too definite. It is somewhat similar to the shear permitted between stirrups, which the Code limits to roughly $10\sqrt{f'_c}$.

ACI 318-77, Section 11.8, provisions for deep flexural members, states that shear strength V_n shall not be taken greater than $8\sqrt{f'_c}b_w d$ when span-to-depth ratio is less than 2. Werner and Dilger [27] report that the tensile strength for the concrete may be equal to $6\sqrt{f'_c}$, and this cracking load can be taken as the shear force which is resisted by the concrete. The ACI Building Code specifies $7.5\sqrt{f'_c}$ as the modulus of rupture of concrete.

Using shear strength of $6\sqrt{f'_c}$ to $8\sqrt{f'_c}$, the shear strength of the dry single key joint was calculated.

$$V_{n_1} = (6\sqrt{6630} \text{ to } 8\sqrt{6630} \times 3 \times 6/1000$$

$$= 8.8 \text{ to } 11.7 \text{ k}$$

Assuming that V_{n_1} includes all shear transfer effects through the shear key section, the shear friction contribution on the remainder of the joint is computed as

$$V_{n_2} = N\mu = 26.0 \times 14/20 \times 0.7 = 12.7 \text{ k}$$
$$V_n = V_{n_1} + V_{n_2} = 22 \text{ to } 24 \text{ k}$$
$$P = 44 \text{ to } 48 \text{ k}$$

The calculated load P varies from 35 to 54 kips for $\mu = 0.7$. This means that each of these methods gives very similar results. The shear friction results with $\mu = 0.4$ and 0.7 were shown in Section 4.2.1a. The calculated values are above the load at which significant slip had occurred. However, the actual maximum load (89 kips) was still substantially higher than the predicted values.

The PCI Bridge Committee [32] and the 1978 AASHTO Interim Bridge Specifications, Article 1.6.25, specify that at time of erection the temporary shear stress carried by the concrete section with unhardened epoxy would be that engaged by the shear key and shall not exceed $2\sqrt{f'_c}$. This would correspond to 6 kips and is about 20% of the value at which first slip was noticed. This seems quite conservative. However, the test results do not give a direct comparison, since the fluid epoxy would lubricate the surface. Such a condition was not checked in these tests.

4.2.2.4 Reinforcement for the key. The reinforcement for the single keys used in the test was proportioned similar to that used by Kashima and Breen [22]. The reinforcement parameter $A_v f_y$ was 1 kip, while the normal compressive force N on the key portion was about 8 kips. Since f_y of the reinforcing wire was much lower than expected, as mentioned in Sec. 2.2.2, the product of $A_v f_y$ was also smaller than intended. However, the key reinforcement had a visible influence on the failure pattern of the test specimen. Splitting and spalling occurred in the plane of the key reinforcement. It is reported [9] that a single key joint in the Kishwaukee River Bridge had a serious crack which caused concrete spalling.

4.2.3 Multiple Key Joint

4.2.3.1 Behavior of the test specimen with a nonepoxied multiple key joint. As shown in Figs. 3.10 and 3.11, a direct shear failure of the multiple keys occurred. Just before the major direct shear failure, occurrence of short diagonal cracks was observed in the multiple keys. Due to the characteristics of direct shear failures, the damage was well-confined within the key portion and it did not extend to other regions (in contrast with the case of a dry single key joint).

The load vs slip curve (Fig. 3.4) was multilinear. The initial portion of the curve which represents nonslip behavior was identical to that of the keyless joint. Slips took place gradually, maintaining much higher stiffness up to the major failure load. The load vs slip relationship and visual observation indicated a progressive failure of the multiple keys after the first slip.

4.2.3.2 Forces acting on multiple keys. Figure 4.3 illustrates forces acting on multiple keys of the specimen with the nonepoxied multiple key joint. The resultant forces were calculated in the same way as described in Sec. 4.2.2.2, assuming perfectly identical geometrical condition for each key. In Fig. 4.3, three phases of the loading are illustrated. Those phases are before-slipping at P = 30 k, after-slipping at P = 58 k (no moment), and before major key failure at P = 96 k. As the load P increases, the resultant force acting on each lug-key increases its magnitude, and the direction of the force vector approaches a vertical line. The effect of the moment due to prestressing and load application in the shear test was taken into account in the calculation of the resultant forces. The moment values shown in Fig. 4.3 are the sum of the moments due to prestressing (as calculated using the prestress forces reported in Table 3.2) and the moments due to the applied load P (as calculated using M = 3/2 P as shown in Fig. 3.3(b)).

Although the existence of moment seemed to have a trivial effect, it might have played some role in continuously progressive softening of the joint stiffness. However, the progressive softening of the joint stiffness may mainly be attributed to the fact that multiple keys cannot be made perfectly identical to each other.

Occurrence of diagonal cracks within the lug keys may be explained using the pattern shown in Fig. 4.3(c). The direction of a diagonal crack may agree with that of a resultant force. Generally, very high compression tends to cause high tensile stress perpendicular to the

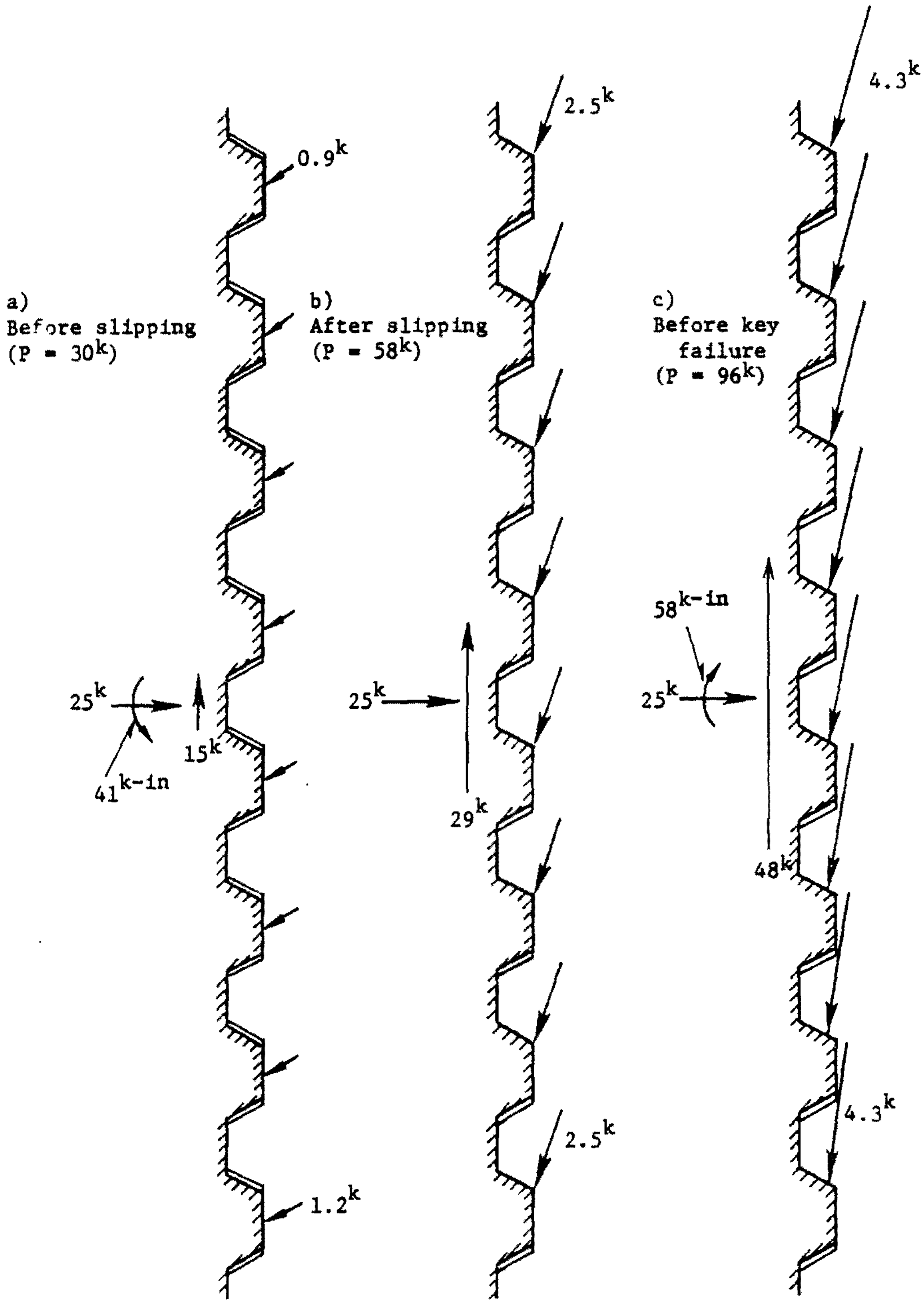


Fig. 4.3 Forces acting on multiple keys (without epoxy)

compressive force due to the effect of Poisson's ratio, resulting in splitting. Maximum shear stresses will occur at the base planes of the keys as a result of distributed loading on the top faces of the keys. Therefore, direct shear failure will take place at the base of each key.

Figure 3.11 shows sheared-off multiple keys in the specimen without epoxy. Small clearances and tight contact action were alternately observed between the top or bottom faces of the mating keys after the test. Therefore, it is considered that the assumption with respect to the force transfer mechanism between the adjacent keys is valid at least for the case of force transfer after-slipping.

4.2.3.3 <u>Direct shear strength</u>. The shear strength of a dry multiple key joint could be calculated in a similar fashion to the procedure used for a single key joint based on a nominal concrete shear strength of 6 f_c to 8 f_c as discussed in Sec. 4.2.2.3 (d).

Direct shear on keys

$$V_{n1} = mvb_wh$$

m = number of keys

v = direct shear strength

b_w = width of keys

h = depth of keys at base

$$\therefore V_{n1} = 8 \times (6\sqrt{7000} \text{ to } 8\sqrt{7000}) \times 3 \times 1.0/1000$$
$$= 12.0 \text{ to } 16.1 \text{ k}$$

Shear friction

$$V_{n2} = N_{\mu} = 24.8 \times 14/20 \times (0.7)$$
$$= 12.2 \text{ k}$$

$$V_n = V_{n1} + V_{n2} = 24.2 \text{ to } 28.3 \text{ k}$$

Bearing check

$$0.85f'_cA = 0.85 \times 7 \times 8 \times 3 \times 0.4$$
$$= 57.1 \text{ k} \quad \underline{\text{O.K.}}$$

$$\therefore P = 2V_n = 48 \text{ to } 57 \text{ k}$$

The calculated load P is shown in Table 4.2 for comparison with the measured load which was much higher than predicted. It is interesting to note that both calculated and measured loads for single and multiple key specimens are in the correct general proportion.

In general, multiple keys are not reinforced.

4.2.3.4 Other aspects of multiple key joint. In practical construction, multiple keys could be one of the most vulnerable parts of precast segments at the time of handling of the segments, which would take place frequently due to the nature of precast segmental construction. In fact, it is reported [35] that in the Long Key Bridge, some of the keys were occasionally chipped or broken off. Concerning this problem, an engineer with Figg & Muller, Inc., reportedly said, "One broken, even two doesn't bother me. More than that we'd have to think about. But we haven't had more than two (out of eighteen)." This remark characterizes the general performance of multiple key joints. Schaijik [35] also reported that the problem of chipped and broken keys was encountered while producing Vail Pass Bridge segments, and chicken wire mesh in the keys cured that problem. In these shear tests, no such problems were experienced.

As to the shape of multiple keys, Schaijik also reported that the original circular corrugations had been replaced by the superior geometry of trapezoidal keys for production and assembling reasons, and that circular corrugations could not always produce a tight fit.

4.2.4 Effect of Epoxy

4.2.4.1 Effectiveness of epoxy. As mentioned in Sec. 1.2.1, the major functions of epoxies are (1) to act as a lubricant, (2) to even out minor irregularities between the mating surfaces during erection, (3) to provide watertightness and durability at the joint, and (4) to transfer shear forces in the cured state. In the shear test, the structural effect of epoxy on the performance of precast segmental joints was phenomenal.

As mentioned in Sec. 3.4.3, all three specimens with epoxied joints, including the no-key joint, attained much higher failure loads than any specimen with a nonepoxied joint. The maximum loads (116 to 134 kips) were similar to those (118 to 134 kips) for the monolithic specimens. As shown in Table 4.1, measured shear strengths of the epoxied joints were in every case higher than the calculated shear strengths of monolithic specimens. The calculated shear strength of the epoxied joints based on shear friction theory but with a coefficient of friction of 1.4 as is assumed for monolithic concrete is given in Table 4.2. The calculated values range from 70 to 73 kips and were much lower than the measured failure loads. This, along with the fact that no slip was noticed in the epoxied joints up to failure loads, led to the conclusion that the specimens with epoxied joints behaved monolithically and failed at their web shear and/or bearing capacities.

As described in Sec. 3.4.2(b), all three specimens with epoxied joints had almost the same load vs relative displacement curves. Table 4.2 indicates the failure loads for the epoxied specimens were the same range and magnitude as the monolithic baseline specimens. Load-slip behaviors of the joints studied herein are summarized in Fig. 4.4. Figure 4.4 confirms that the epoxy enabled all joint types to act monolithically and much superior to the dry joint specimens.

4.2.4.2 <u>Areas for further studies</u>. Regarding the basic characteristics of the epoxy bonding agent, Hugenschmidt [36] reported as follows:

> The properties of epoxies are greatly influenced by variations in temperature. Because of this sensitivity to the temperature, the testing of epoxies is expensive and time consuming. The short-term strengths (compression, flexure, shear strength, lap shear strength, and modulus of elasticity) are usually deceptively high. Furthermore, they can easily give the erroneous impression that the mechanical strength of an epoxy system is always greater than that of the concrete to be bonded. If the concrete is being bonded under mild conditions, the requirement "failure in concrete" is easy to fulfill under most prevailing stresses. The adhesive strength of the epoxy can be assumed to be greater than the ultimate strength of the concrete and is therefore not a governing criterion. The important criteria of an appropriate epoxy adhesive are creep deformation, heat stability, and moisture resistance.

His article strongly suggests the need for investigation of long-term behavior of epoxied joints.

The relationship between the thickness of the epoxy layer in the joint and the segment size in the model test may not be exactly similar to that in the prototype construction, since the same amount of temporary post-tensioning stress is used in both cases. The epoxy layer

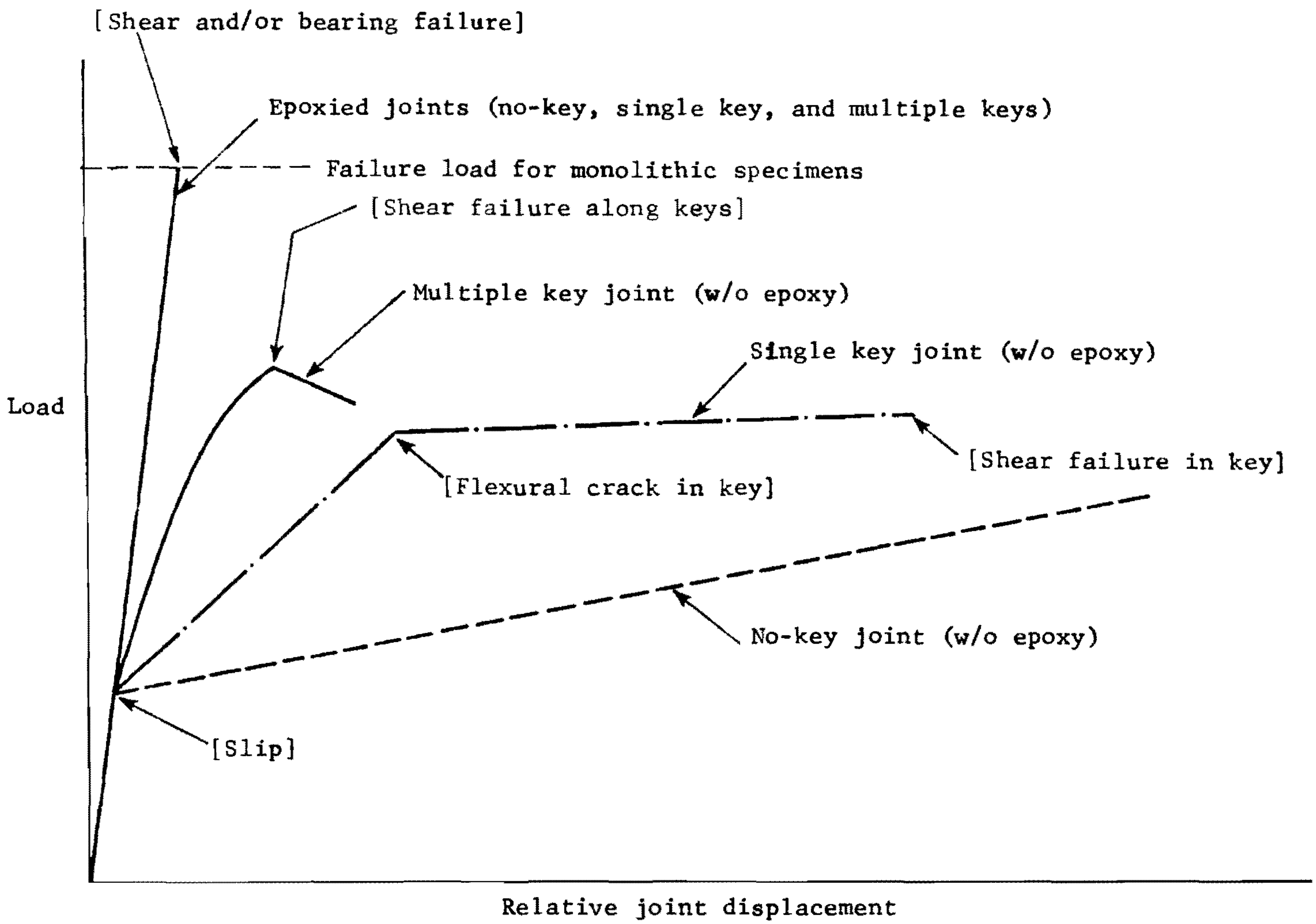


Fig. 4.4 Comparison of behavior of joints

in the joint of the model test specimen may tend to be relatively thicker. In this respect, tests using prototype-size specimens might be needed.

Kashima and Breen [30] have pointed out that many epoxies furnished as suitable for jointing concrete segments in fact are unsuitable. The suitability of specific formulations should be checked using simple tests but with surface conditions and ambient factors typical of the proposed application.

4.3 Appraisal of Types of Joint

The overall findings from these limited exploratory tests on shear strength of joints in precast segmental bridges are condensed in Fig. 4.4.

In terms of the maximum loads developed, all epoxied joints behaved similarly and developed loads equal to those carried by the monolithic specimens. Among the nonepoxied joints, the multiple keys showed higher strength, though the maximum value was significantly lower than those of the epoxied joints. The nonepoxied single key joint carried less load than the load developed by the multiple keys. As expected, the keyless joint without epoxy carried the lowest load. The loads at initial slip for nonepoxied joints were almost identical, while no significant slip was observed in the epoxied joints. Comparison of absolute values of the maximum loads between nonepoxied single and multiple key joints may not be important, since those values could be changed by designing a key configuration differently, especially by increasing or decreasing the shear key area-to-web section area ratio. In general, use of multiple keys assures more shear key area and results in higher shear strength of the joint. In a single key configuration, there seems to be a limit on strength increase by increasing the key area.

A more important consideration for nonepoxied joints is the behavior of the joint as indicated by the vertical load vs slip relationship. Figure 4.4 indicates a clear superiority of the multiple key dry joints over the single key dry joints. It appears that the single key joints should not be used without an epoxy bonding agent, as specified by the Precast Segmental Box Girder Bridge Manual [2]. When the shear load is very small and corrosion resistance and durability do not require its use, the epoxy might be omitted. It is clear that use of multiple keys improves the overall performance of the dry joints. However, application of an adequate epoxy bonding agent provides much better assurance.

Epoxy which does provide structural assistance is available for a modest cost. The other major benefits of the epoxy bonding agent such as water tightness and durability of the joint are automatically enhanced by its use. Therefore, the use of epoxy is strongly recommended. Use of an adequate epoxy bonding agent allows use of the single large shear key which might be advantageous in some cases.

Even though this program was limited it was apparent that from the viewpoint of construction simplicity there are important differences between single and multiple keys. The single large key requires placement of key reinforcement. It is more difficult to conceal and, hence, is less aesthetic. The small multiple keys have no need for key reinforcement and can be easily concealed. They do require somewhat more complicated forms and may be more fragile and prone to handling damage.

As mentioned in Sec. 1.1, temperature and weather limitations regarding mixing and placing epoxy jointing material may be one of the possible disadvantages of precast segmental construction. On this subject, Gentilini and Gentilini [37] reported their experience as follows:

> In the wintertime, the surface to be bonded was electrically heated using armored electric cables which were buried in the precast segment at a depth of 1-in. from the surfaces to be bonded. This heating procedure was adopted after some negative experiments with traditional systems where heating is provided from the outside of the concrete.

A strictly controlled program of curing epoxies was utilized in the construction of the Olympic Stadium in Montreal. Construction proceeded successfully under winter conditions. The technology exists to utilize epoxies correctly, although many examples of misuse have been reported in the relatively brief American history of segmental bridges.

CHAPTER 5

CONCLUSIONS

All conclusions in this study must be qualified because of the limited test program undertaken. Only one reliable epoxy was used and single model specimens were used under a single loading condition. However, within that context the following conclusions are warranted:

(1) Load vs relative joint displacement relationship

Each type of joint configuration showed a distinct load vs joint slip relationship (Fig. 4.4). In nonepoxied specimens, the load vs slip curve was bilinear for the keyless joint, trilinear for the single key joint, and multilinear for the multiple key joint. The relative loads at given slips were substantially higher for the multiple key joint. In epoxied specimens, no significant slip at the joints occurred up to the major failure load.

The load vs relative joint displacement relationship of all specimens without epoxy were almost identical to each other until initial slips occurred. Thus, contributions of shear friction and of keys to the resistance to initial slip were not additive.

(2) Shear friction

The shear friction provisions of ACI 318-77 overestimated the slip load of the specimens with nonepoxied joints unless coefficients of friction were reduced to 0.55 to 0.61 from conventional values which vary from 0.7 to 1.0. Prestressing forces were considered to be additive to the reinforcement parameter ρf_y.

(3) Nonepoxied single key joint

In the specimen with the nonepoxied single key joint, flexural cracks were first observed at the junction of the top face of the male single key and the end face of the segment. After development of the flexural cracks, major shear failure occurred in the base plane of the male key, accompanied by splitting cracks in the key reinforcement planes.

Corbel provisions of ACI 318-77 and the PCI Design Handbook or direct shear strength calculation based on $6\sqrt{f'_c}$ to $8\sqrt{f'_c}$ shear stress gave similar values of shear strength for the single key joint. However, these calculated shear strengths were only 60% of the actual maximum shear force.

(4) Nonepoxied multiple key joint

In the specimen with the nonepoxied multiple keys, a direct shear failure of the multiple keys took place. Again, the calculated load using nominal concrete shear strength of $6\sqrt{f'_c}$ to $8\sqrt{f'_c}$ was only 60% of the actual maximum load.

(5) Epoxy

The effect of epoxy on the performance of precast segmental joints was phenomenal. All three specimens with epoxied joints, including the keyless joint, acted monolithically, carrying loads as high as the monolithic no-joint specimens. The measured failure loads of the epoxied specimens were 60 to 80% higher than the calculated shear strength of the joints based on a shear friction theory with the coefficient of friction assumed as 1.4, as used for fully monolithic concrete.

(6) Appraisal of types of joint

The results indicated that single key joints should always be used with epoxy bonding agents. If nonepoxied joints are to be used, the use of multiple keys improves the overall performance of the joints. However, application of an epoxy bonding agent provides much better total assurance, and, therefore, it is highly desirable.

(7) Design procedures

(a) Precast segmental joints without epoxy will be controlled by slip in the joint. A conservative design procedure is to utilize ACI-AASHTO shear friction provisions but with the value of μ taken as 0.5.

The ultimate strength of both single key and multiple key specimens can be conservatively estimated by using a nominal shearing stress of $8\sqrt{f'_c}$ on the key shear area.

(b) Precast segmental joints with a properly controlled epoxy jointing material will behave like monolithically cast concrete. Normal ACI-AASHTO provisions for determining flexural, bearing, and shear strength are applicable to the properly cured joint.

(8) Further studies

There is a need for investigation of long-term behavior of epoxied joints. Additional tests using prototype-size specimens should be run. In addition, tests of specimens with various epoxies and jointing conditions, with nonepoxied joints with various key shapes, and with bonded tendons might be useful. A construction age test series should be run with epoxy joints before the epoxy solidifies. The joint behavior should be studied under reversed and fatigue loads.

In any further study, an improved joint slip measurement system should be used in place of the crude system used in this study.

REFERENCES

1. Podolny, W., "An Overview of Precast Prestressed Segmental Bridges," PCI Journal, Vol. 24, No. 1, Jan.-Feb. 1979, pp. 56-87.

2. Precast Segmental Box Girder Bridge Manual, Post-Tensioning Institute and Prestressed Concrete Institute, Phoenix and Chicago, 1978, 116 pp.

3. Podolny, W., and Ray, G. K., "Concrete Segmental Bridges," Constructor, Vol. 62, No. 8, Aug. 1980, pp. 34-37.

4. Mathivat, J., "Récents Développements des Ponts à Voussoirs Prefabriques," IABSE Surveys S-2/77, International Association for Bridge and Structural Engineering, May 1977, 24 pp.

5. Muller, J., "Ten Years of Experience in Precast Segmental Construction," PCI Journal, Vol. 20, No. 1, Jan.-Feb. 1975, pp. 28-61. And, Reader's Comments (by Bender, B., and Libby, J. R.), Sept.-Oct. 1975, pp. 81-85.

6. Bridges, C. P., and Coulter, C. S., "Geometry Control for the Intercity Bridge," PCI Journal, Vol. 24, No. 3, May-June 1979, pp. 113-125.

7. Leonhardt, F., Zellner, W., and Svensson, H., "Die Spannbeton-Schrägkabelbrücke über den Columbia zwischen Pasko und Kennewick im Staat Washington, USA," Beton-und Stahlbetonbau, Vol. 75, No. 2, Feb. 1980, pp. 29-36.

8. Figg, E. C., "The Long Key Bridge--Segmental Bridge Design in the Florida Keys," Concrete International, Vol. 2, No. 8, Aug. 1980, pp. 17-22.

9. Anon., "Epoxy Blamed for Crack in Bridge," Engineering News-Record, July 26, 1979, p. 13.

10. Stone, W. C., "Design Criteria for Post-Tensioned Anchorage Zone Tensile Stresses," Ph.D. dissertation, The University of Texas at Austin, Jan. 1980.

11. Birkeland, P. W., and Birkeland, H. W., "Connections in Precast Concrete Construction," ACI Journal, Vol. 63, No. 3, March 1966, pp. 345-367.

12. Mast, R. F., "Auxiliary Reinforcement in Concrete Connections," Journal of the Structural Division, ASCE, ST6, June 1968, pp. 1485-1504.

13. ACI Committee 318, Building Code Requirements for Reinforced Concrete (ACI 318-77), American Concrete Institute, Detroit, 1977, 103 pp.

14. ACI Committee 318, Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-77), American Concrete Institute, Detroit, 1977, 132 pp.

15. Mattock, A. H., "Design Proposals for Reinforced Concrete Corbels," PCI Journal, Vol. 21, No. 3, May-June 1976, pp. 18-42.

16. Mattock, A. H., and Hawkins, N. M., "Shear Transfer in Reinforced Concrete--Recent Research," PCI Journal, Vol. 17, No. 2, March-April 1972, pp. 55-75.

17. American Association of State Highway and Transportation Officials, Standard Specifications for Highway Bridges, Twelfth Edition, U.S. Government Printing Office, Washington, 1977.

18. Franz, G., "Versuche über die Querkraftaufnähme in Fugen von Spannbetonträgern aus Fertigteilen," Beton- und Stahlbetonbau, Vol. 54, No. 6, June 1959, pp. 137-140.

19. Jones, L. L., "Shear Tests on Joints between Precast Post-tensioned Units," Magazine of Concrete Research, Vol. 11, Mar. 1959, pp. 25-30.

20. Gaston, J. R., and Kriz, L. B., "Connections in Precast Concrete Structures--Scarf Joints," PCI Journal, Vol. 9, No. 6, June 1964, pp. 37-59.

21. Moustafa, S. E., "Ultimate Load Test of a Segmentally Constructed Prestressed Concrete I-Beam," PCI Journal, Vol. 19, No. 4, July-Aug. 1974, pp. 54-75.

22. Kashima, S., and Breen, J. E., "Construction and Load Tests of a Segmental Precast Box Girder Bridge Model," Research Report No. 121-5, Center for Highway Research, The University of Texas at Austin, Feb. 1975, 261 pp.

23. Hofbeck, J. A., Ibrahim, I. O., and Mattock, A. H., "Shear Transfer in Reinforced Concrete," ACI Journal, Vol. 66, No. 2, Feb. 1969, pp. 119-128.

24. Kriz, L. B., and Raths, C. H., "Connections in Precast Concrete Structures--Strength of Corbels," PCI Journal, Vol. 10, No. 2, Feb. 1965, pp. 16-47.

25. Mattock, A. H., Johal, L., and Chow, H. C., "Shear Transfer in Reinforced Concrete with Moment or Tension Acting across the Shear Plane," PCI Journal, Vol. 20, No. 4, July-Aug. 1975, pp. 76-93.

26. PCI Design Handbook, Prestressed Concrete Institute, Chicago, 1978.

27. Werner, M. P., and Dilger, W. H., "Shear Design of Prestressed Concrete Stepped Beams," PCI Journal, Vol. 18, No. 4, July-Aug. 1973, pp. 37-49.

28. Zeck, U. I., "Joints in Large Panel Precast Concrete Structures," Seismic Resistance of Precast Concrete Panel Buildings, Report No. 1, Department of Civil Engineering, Massachusetts Institute of Technology, Jan. 1976, 174 pp.

29. Kupfer, H., Gückenberger, K., and Daschner, F., "Versuche zum Tragverhalten von segmentaren Spannbetonträgern," Deutscher Ausschuss fur Stahlbeton, Heft 335, 1982.

30. Kashima, S., and Breen, J. E., "Epoxy Resins for Jointing Segmentally Constructed Prestressed Concrete Bridges," Research Report No. 121-2, Center for Highway Research, The University of Texas at Austin, Aug. 1974.

31. Gallaway, T. M., "Precasting of Segmental Bridges," ACI Journal, Vol. 72, No. 10, Oct. 1975, pp. 566-572.

32. PCI Bridge Committee, "Tentative Design and Construction Specifications for Precast Segmental Box Girder Bridges," PCI Journal, Vol. 20, No. 4, July-Aug. 1975, pp. 34-42.

33. American Association of State Highway and Transportation Officials, Standard Specification for Highway Bridges, Interim Specifications, Bridges, U.S. Government Printing Office, Washington, 1978.

34. Ferguson, P. M., Reinforced Concrete Fundamentals, 4th Edition, John Wiley & Sons, New York, 1979, 724 pp.

35. Anon., "Record-Length Project Designed for Utter Simplicity," Engineering News-Record, April 24, 1980, pp. 30-32; and, ENR Letters, August 7, 1980, p. 10.

36. Hugenschmidt, F., "Epoxy Adhesives in Precast Prestressed Concrete Construction," PCI Journal, Vol. 19, No. 2, Mar.-Apr. 1974, pp. 112-124.

37. Gentilini, B., and Gentilini, L., "Precast Prestressed Segmental Elevated Urban Motorway in ItalyP," PCI Journal, Vol. 20, No. 5, Sept.-Oct. 1975, pp. 26-43.