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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.

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by

K. Koseki and J. E. Breen

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Reinforced and Prestressed Concrete

Conducted for

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PREFACE

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.

SUMMARY

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.

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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 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.

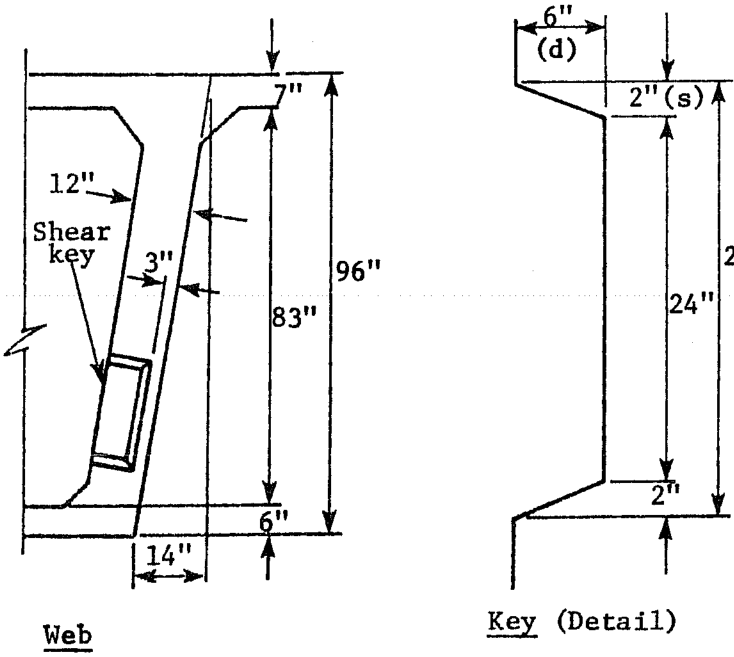
--Frequent loading and unloading of segments, with the risk of damage.

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

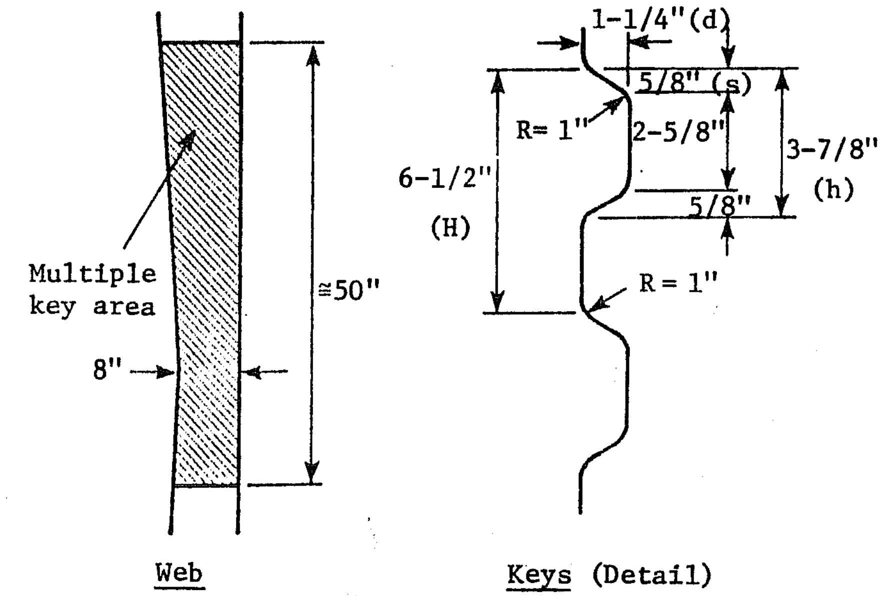
1.2.1 Shear Keys and Epoxy Bonding Agent. The joints between the precast segments are of critical importance in segmental bridge construction. They must have high strength to transfer shear. If tendons pass through the joints, then they must have assured durability in order to protect the tendons against corrosion. In addition,

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

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

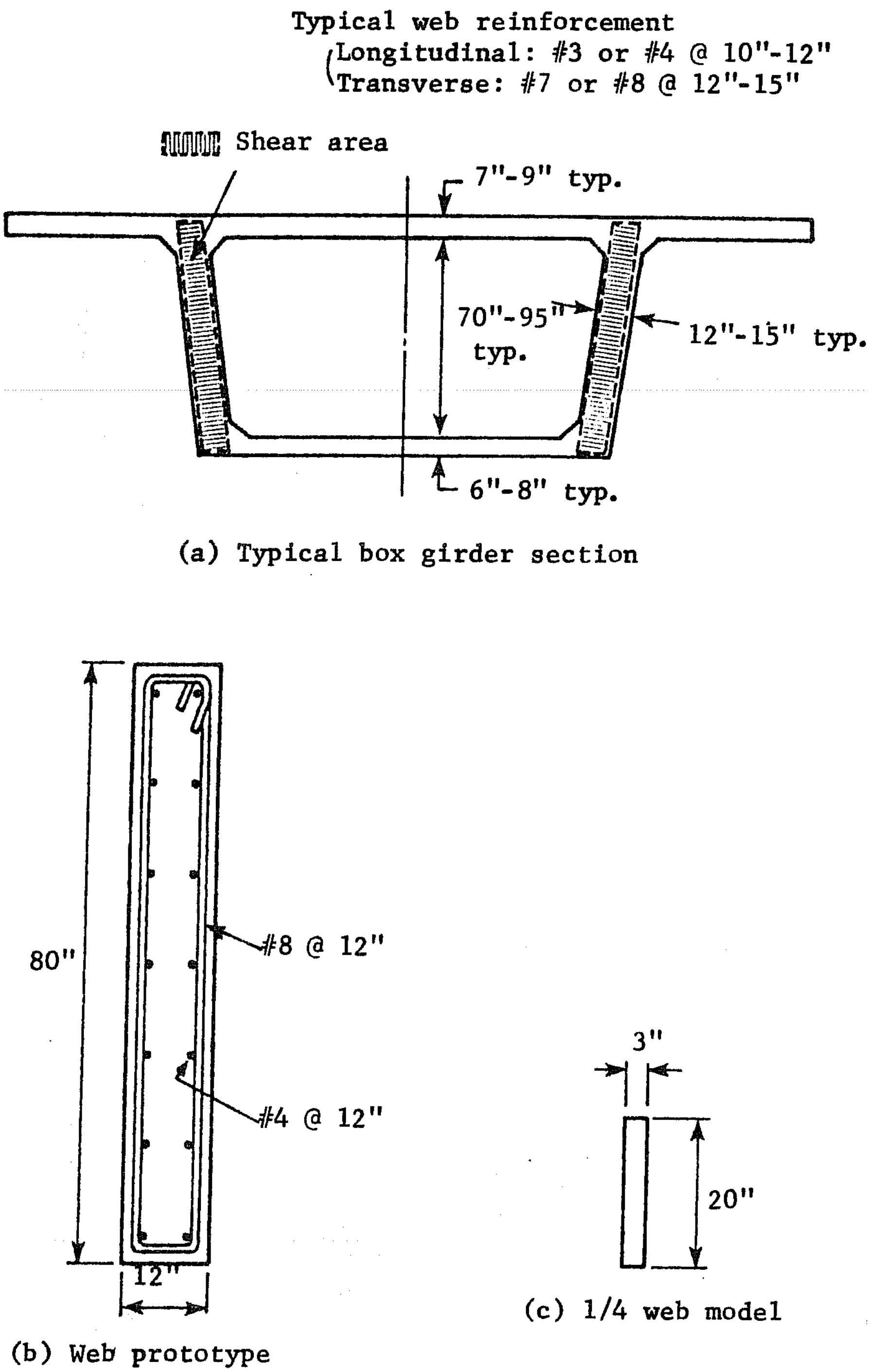
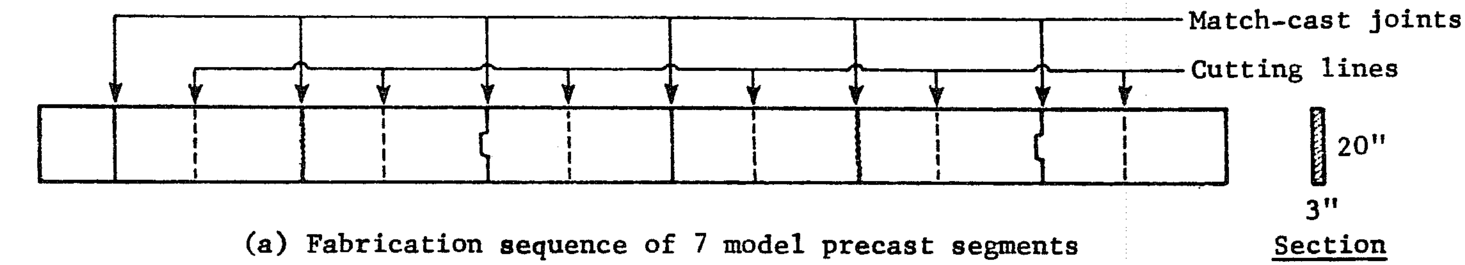
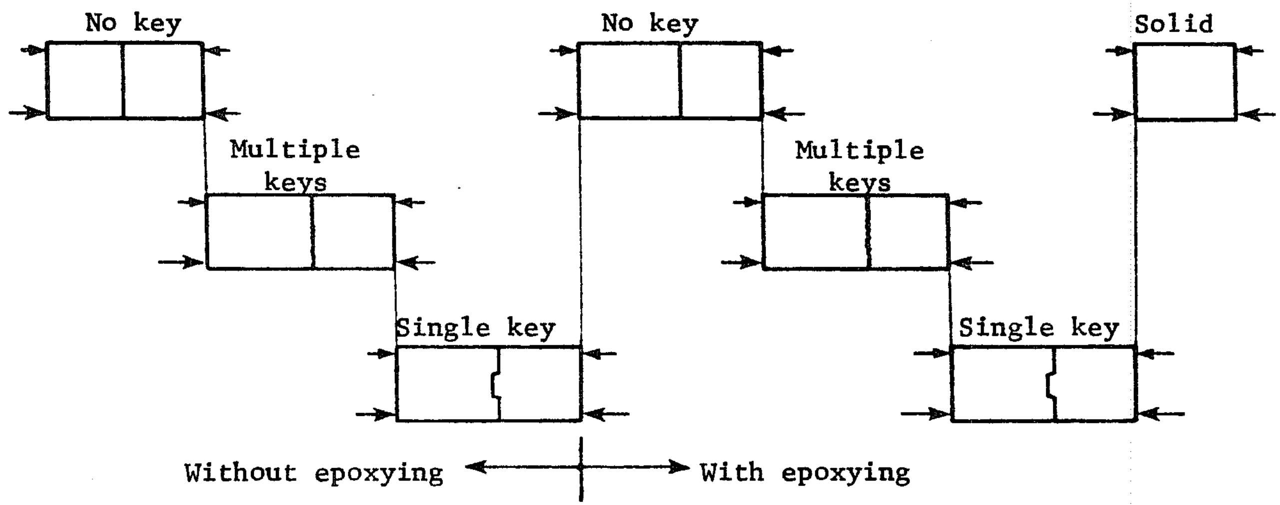


Fig. 1.3 Web model for the tests



(a) Fabrication sequence of 7 model precast segments



(b) Test specimens

Fig. 1.5 Model precast segments and test specimens

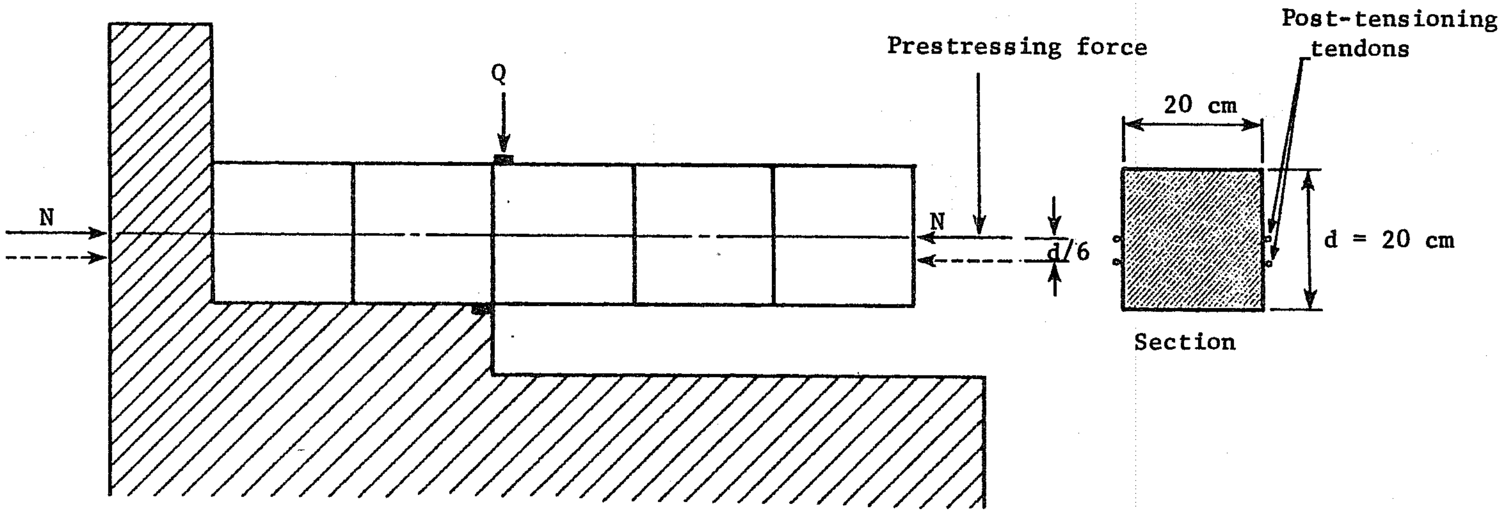


Fig. 1.6 Shear test on joints of precast segmental beams by Franz (Ref. 18)

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 <u>Test by Gaston and Kriz at Portland Cement Association</u> [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 <u>Test by Moustafa at Concrete Technology Corp., Washington</u> [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

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.

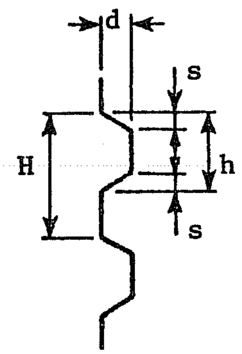
Design of the key in single key joints may be considered to be somewhat analogous to that of corbels. Among the references related to the corbel design are Refs. 13, 15, 16, 23, 24, 25, 26, and 27.

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

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

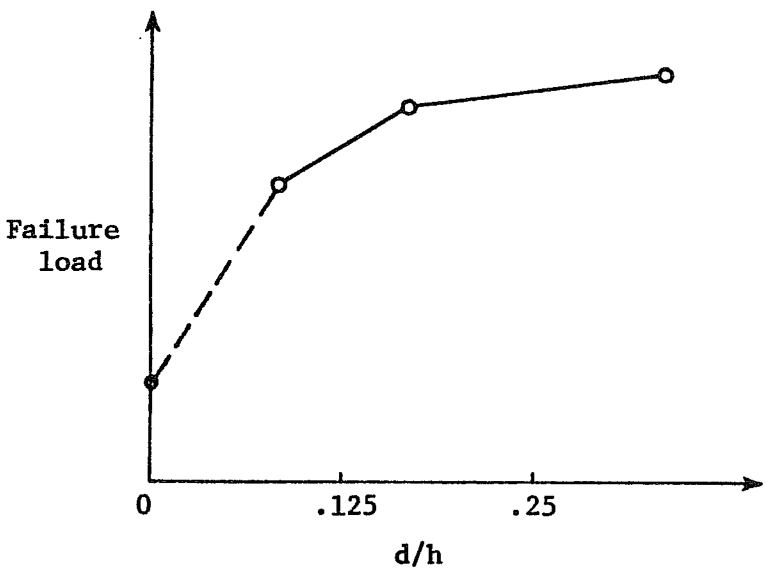


Fig. 1.9 Effect of shear key height to depth ratio (Ref. 28)

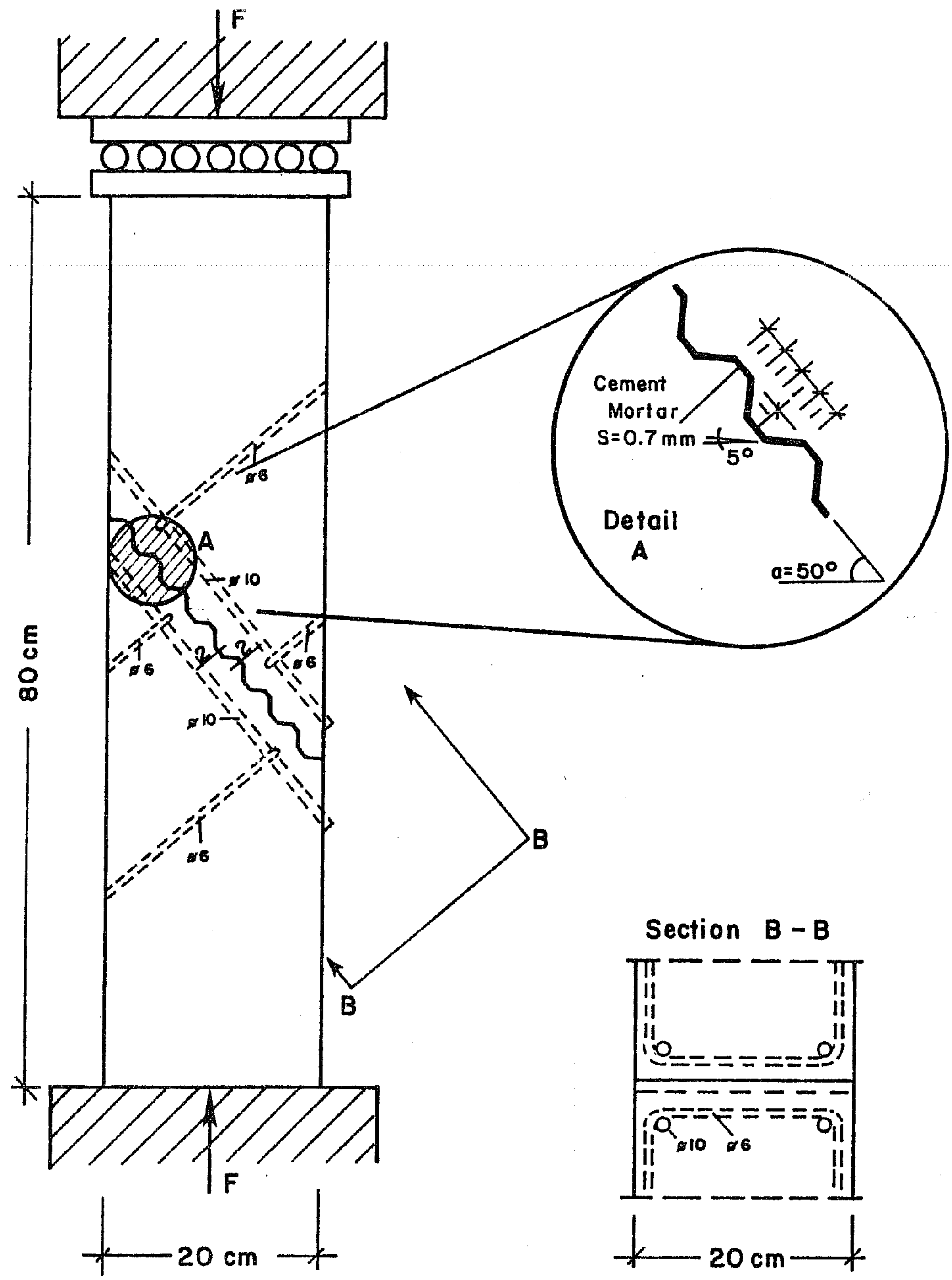


Fig. 1.10 Concrete prism specimen (from Ref. 29)

CHAPTER 2

TEST SPECIMEN

2.1 Specimen Dimensions

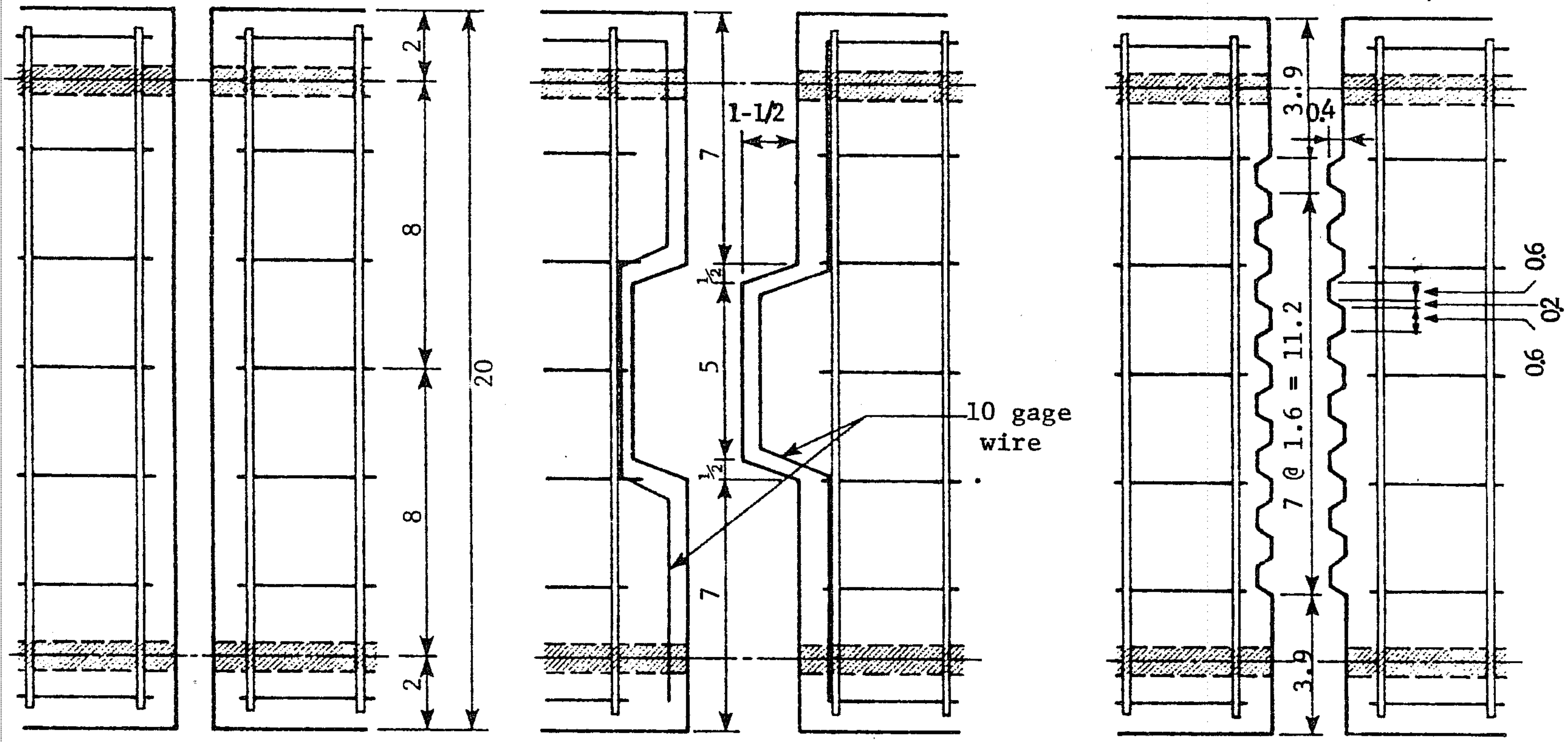
Figure 2.1 shows the general profile of the model precast segment. Dimensions of the segments were 3 in. (width) x 20 in. (depth) x 48 in. (length), except for the first segment whose length was 20 in.

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.

(Unit: inches)



(Key area/web area = .3, d/s = 3/1, d/h = .25)

(h/H = .63, d/s = 2/1, d/h = .4)

a) No-key

b) Single key

c) Multiple keys

Fig. 2.2 Configuration of joints

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

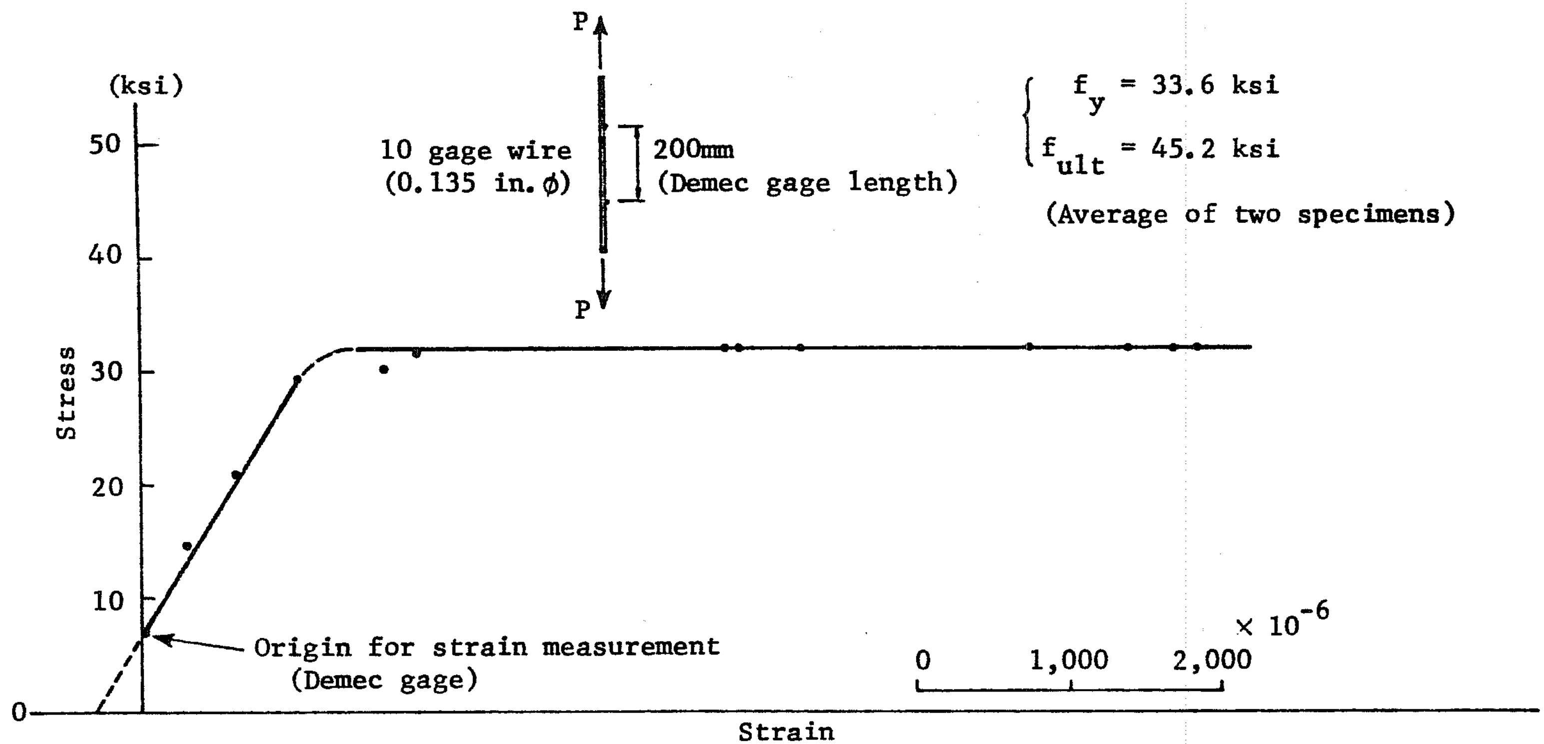


Fig. 2.4 Stress-strain relationship of 10 gage wire

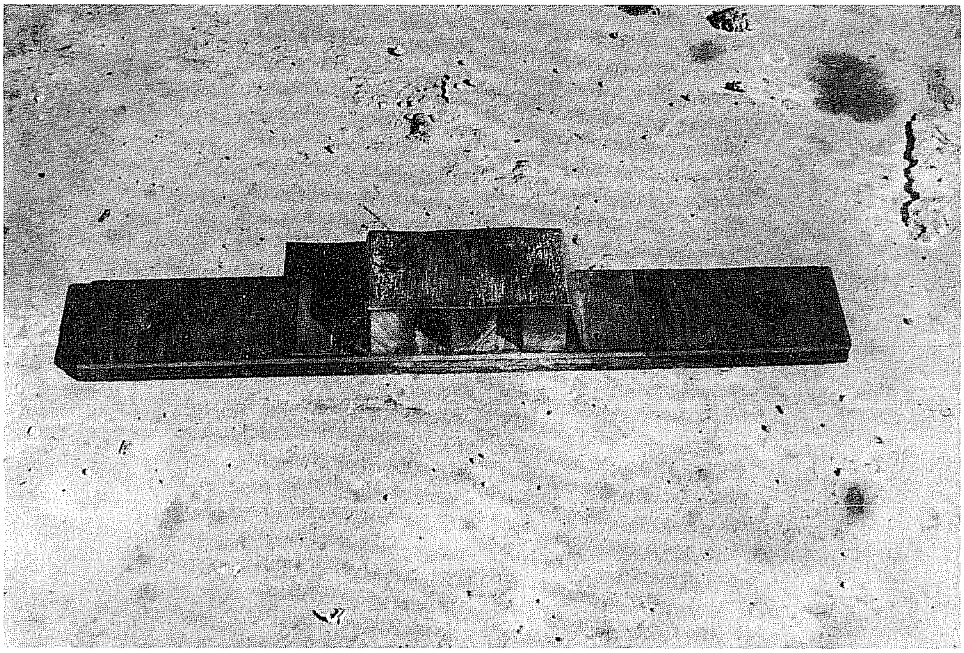
other with 2 in. x 2 in. As seen in Fig. 2.5, both specimens gave similar development curves of epoxy strength. The shear test for the epoxied concrete specimens was planned to be carried out when the epoxy bonding agent was considered to have developed its strength sufficiently. The epoxy bonding agent 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.

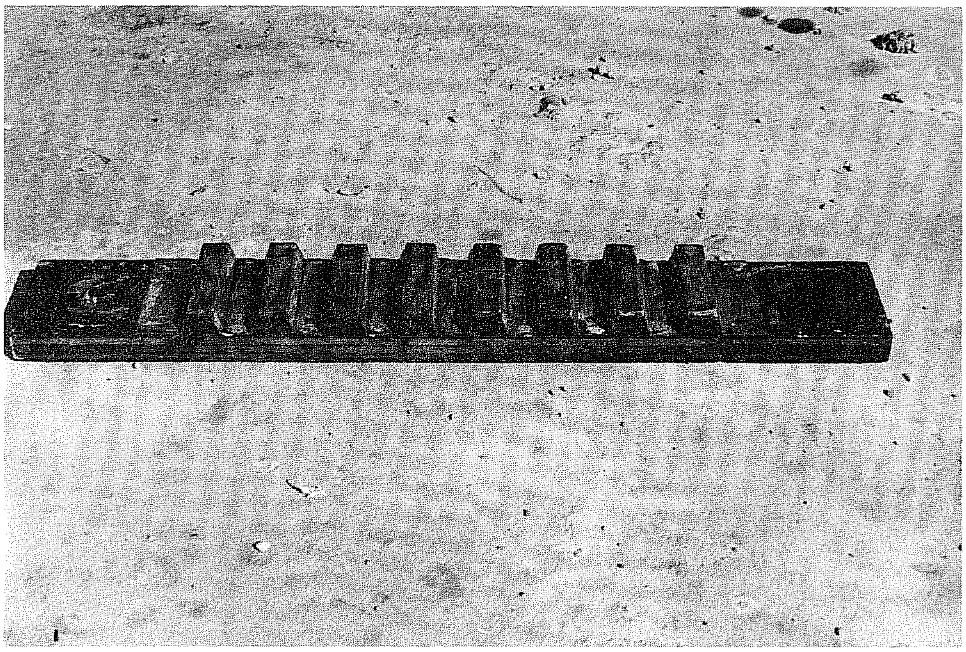
The following is an extract from "Texas Highway Department Special Specification Item 2131 Epoxy Bonding Agent," which is based upon the work done by Kashima and Breen [20] at The University of Texas at Austin:

> The epoxy material shall be of two components, a resin and a hardener, meeting the following requirements:
>
> | | | |
> |---|---|---|
> | a. | Pot life | 90 minutes min. at $68^{o}F$ |
> | b. | Compressive strength | 6000 psi min. |
> | c. | Tensile strength | 2000 psi min. |
> | d. | Specific gravity | 70 to 120 lbs/cu. ft. |
> | e. | Viscosity at $68^{o}F$ | 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.



(a) Single key



(b) Multiple keys

Fig. 2.6 End forms

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).

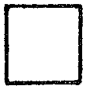
--Cutting and grinding of sheath at end face with a hand saw and a hand grinder.

--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.

TABLE 2.2 CONCRETE STRENGTH AT 7 DAYS

Casting Sequence	Segment	f'_{c7}(psi) (Coefficient of variance) Each (Ave. of 3-ϕ3"×6")	f'_{c7}(psi) (Coefficient of variance) 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-

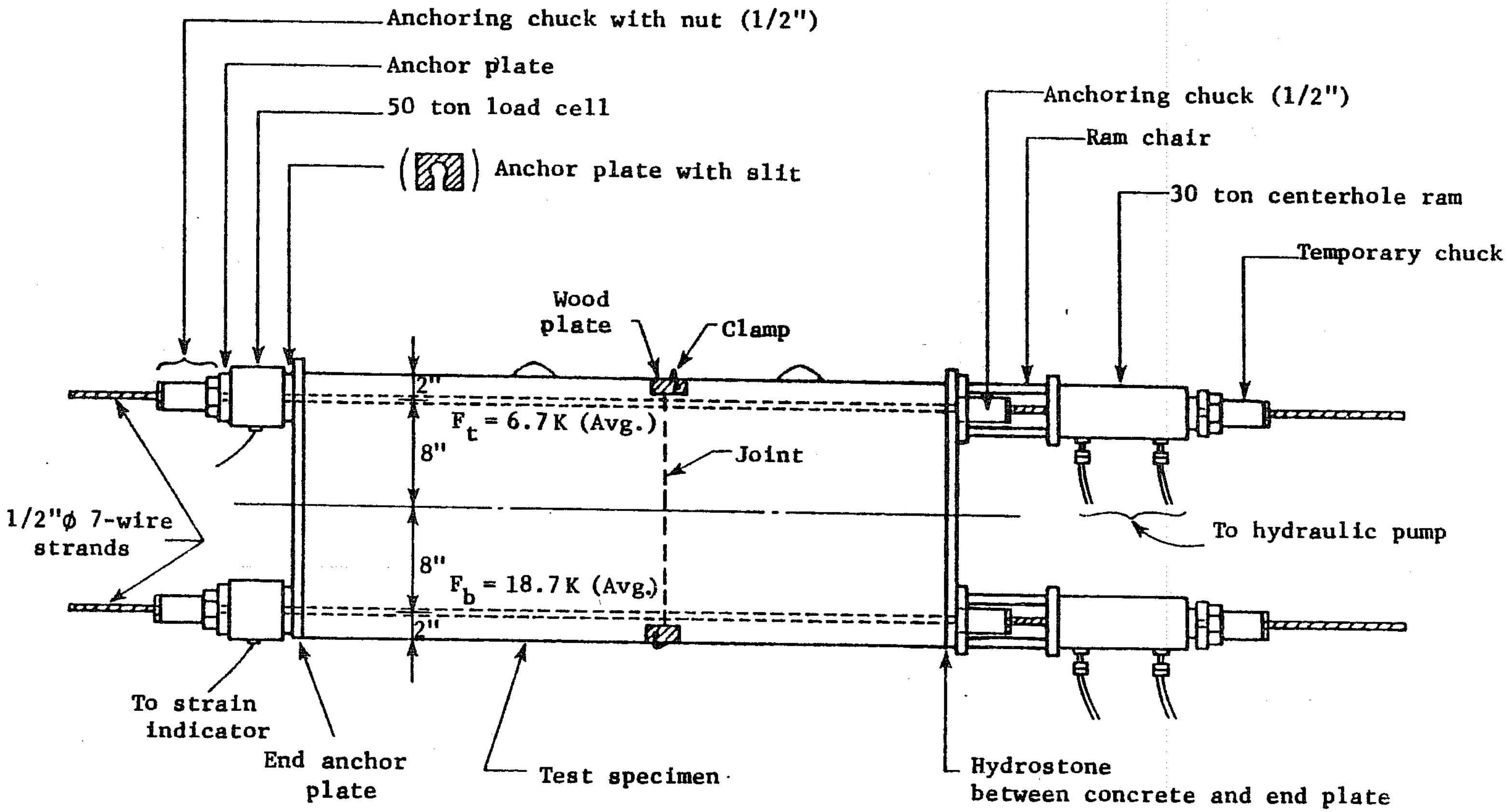


Fig. 3.1 Post-tensioning arrangement

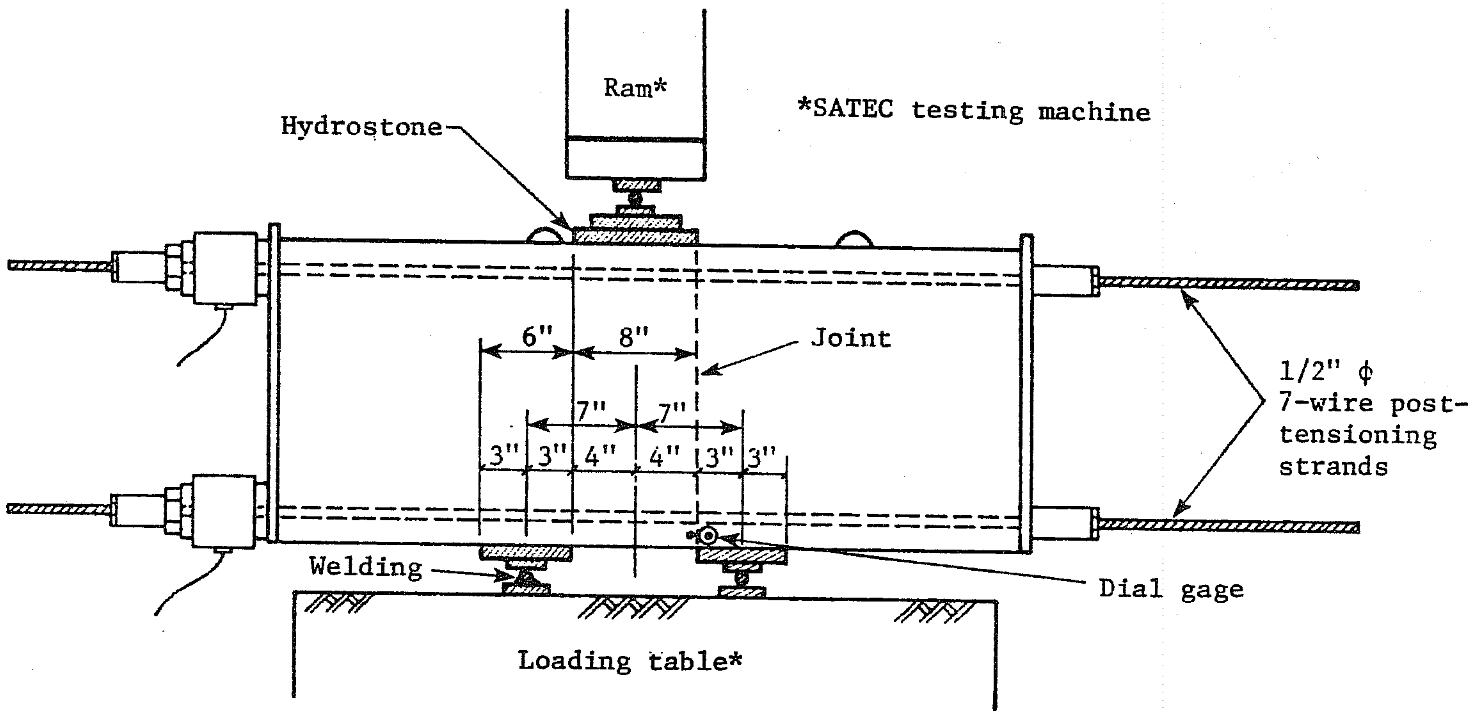


Fig. 3.2 Shear test arrangement

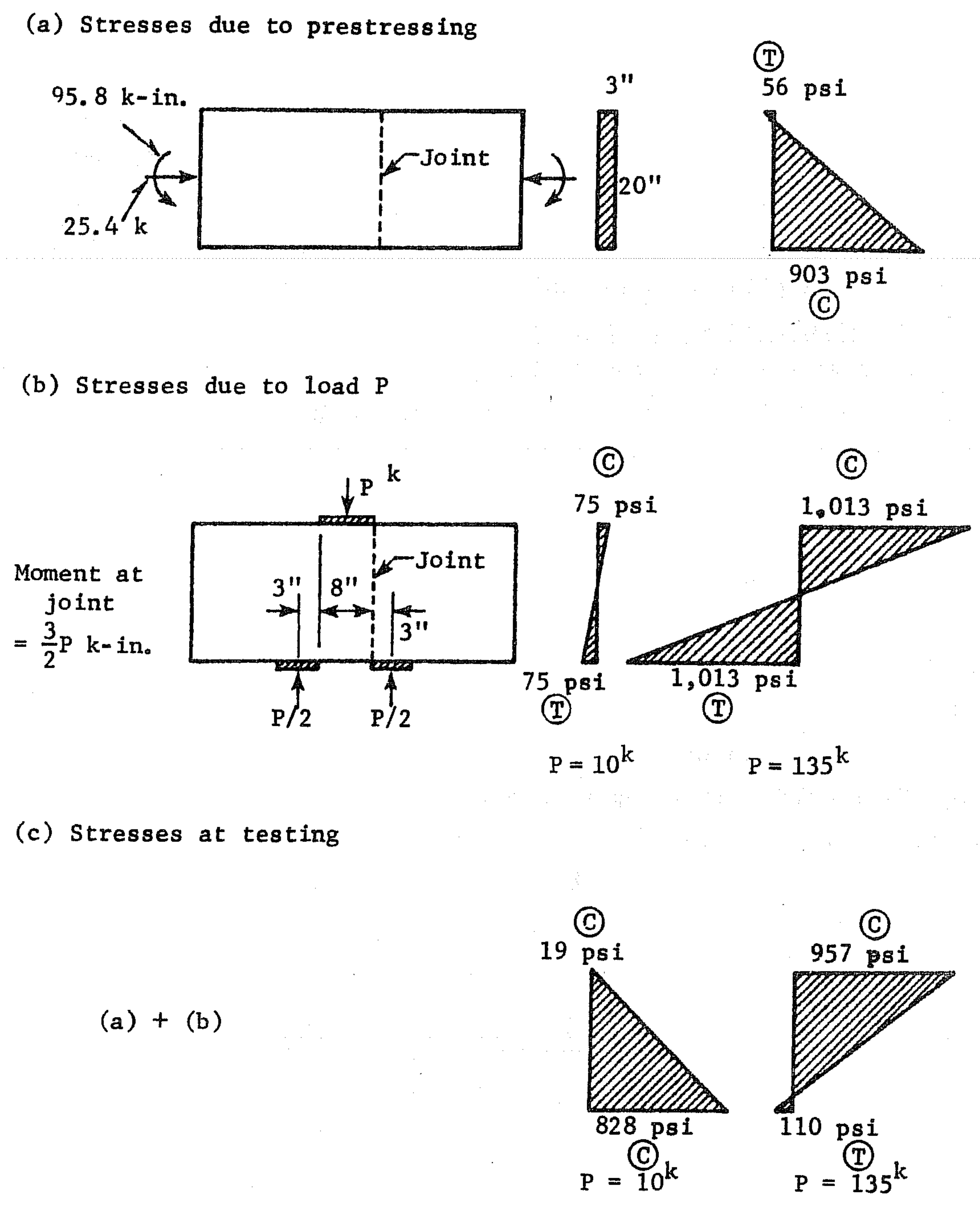


Fig. 3.3 Stresses at joint

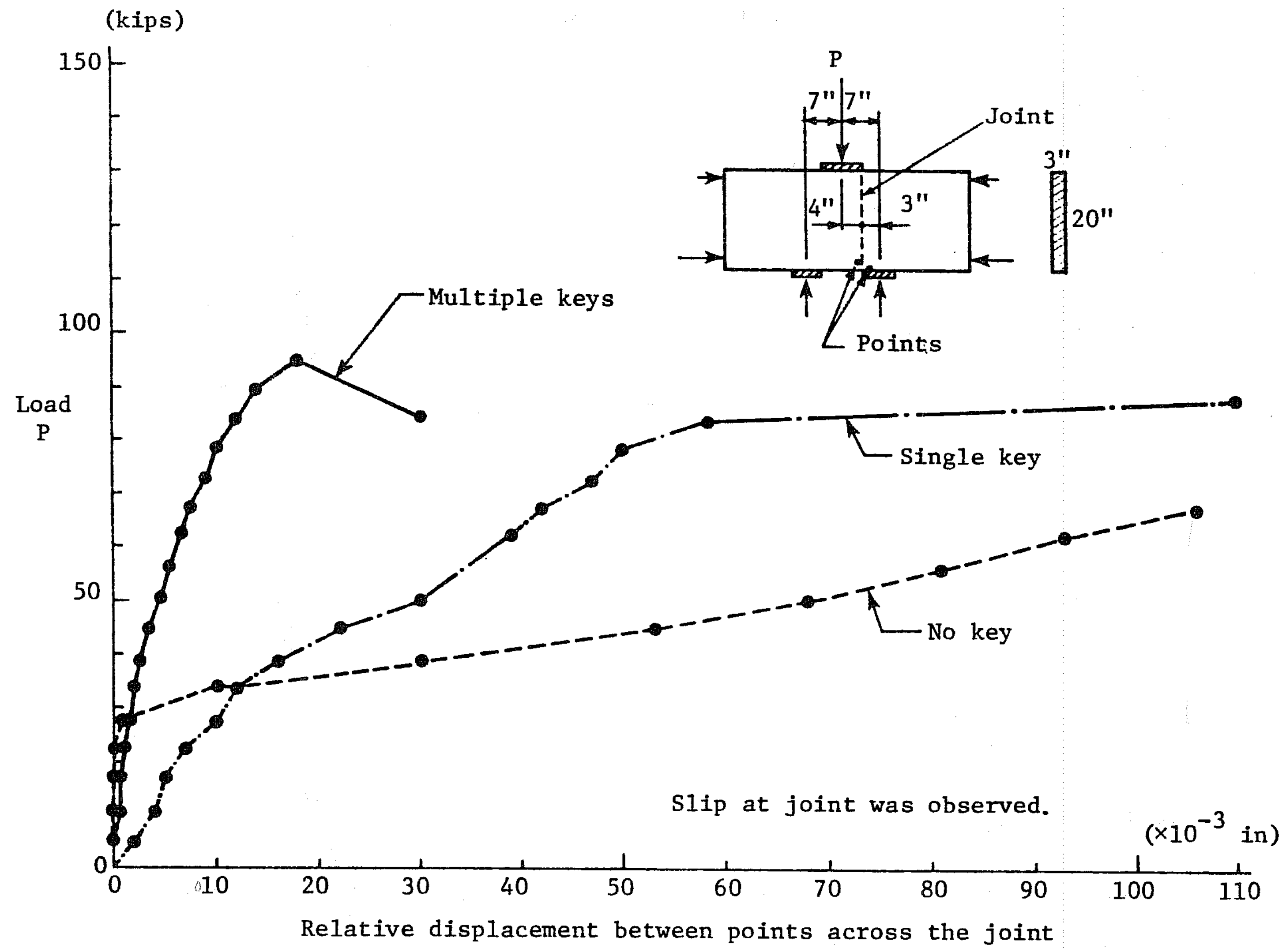


Fig. 3.4 Load vs "slip at joints" without epoxying

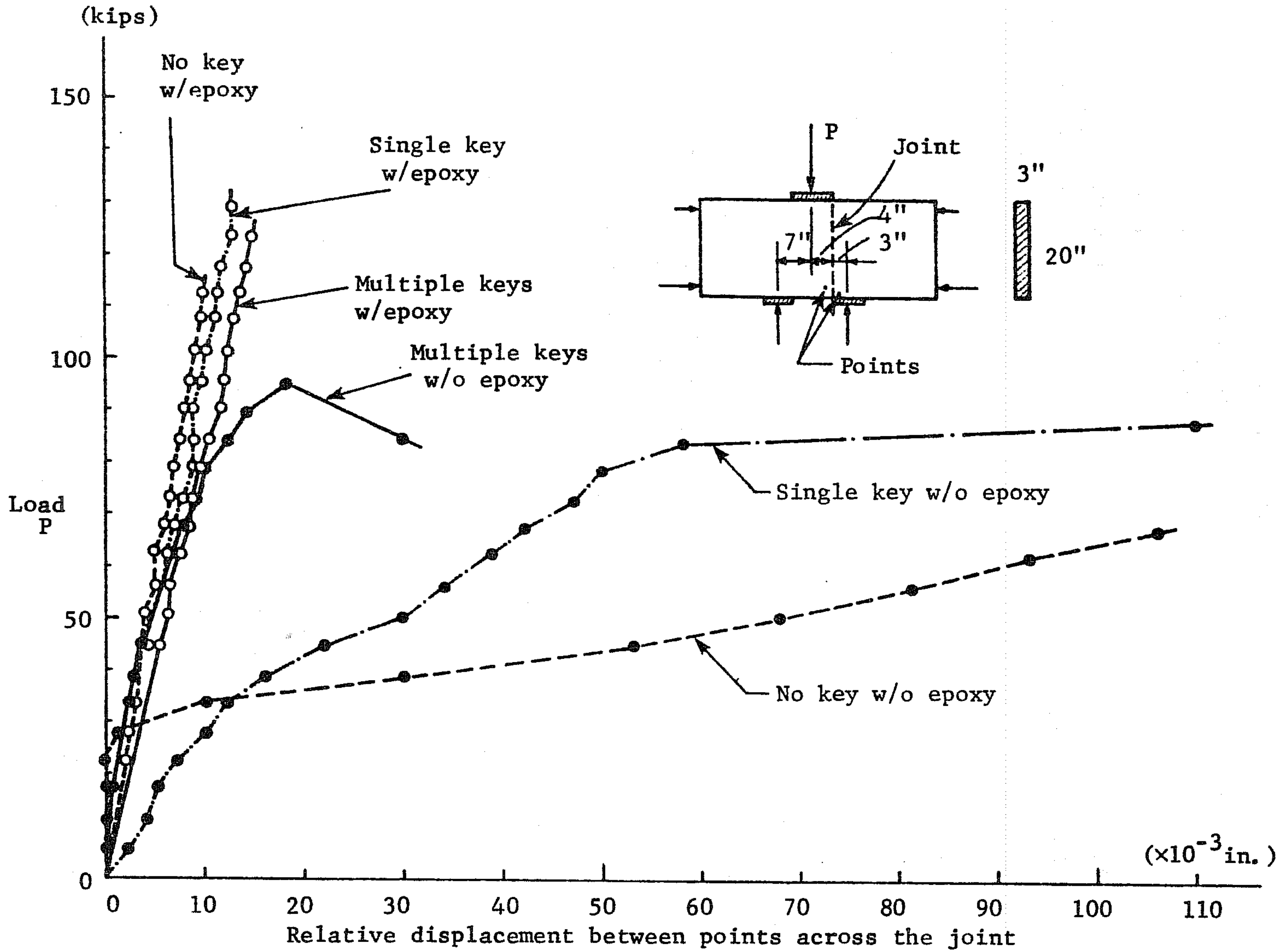


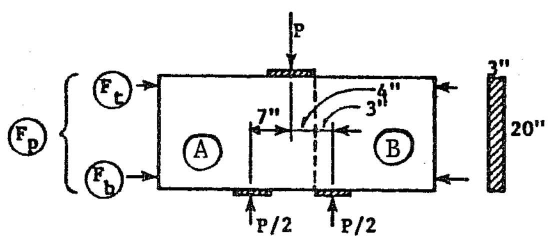
Fig. 3.6 Comparison of the results with and without epoxying

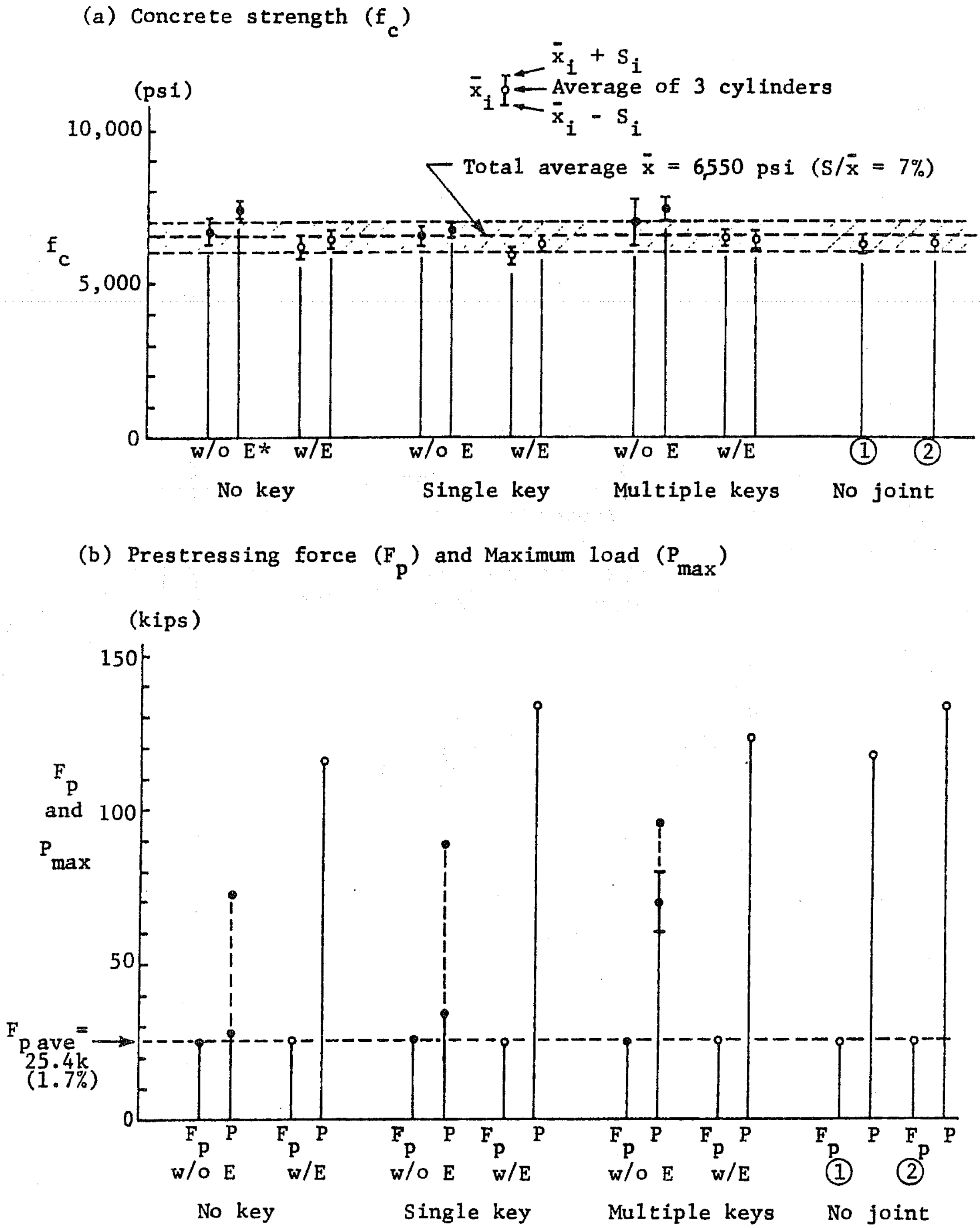
TABLE 3.2 CONCRETE STRENGTH, PRESTRESSING FORCE AND MAXIMUM LOAD

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

f_c = Average of 3 cylinders. *Slipping load.

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





*w/o E = without epoxy; w/E = with epoxy

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

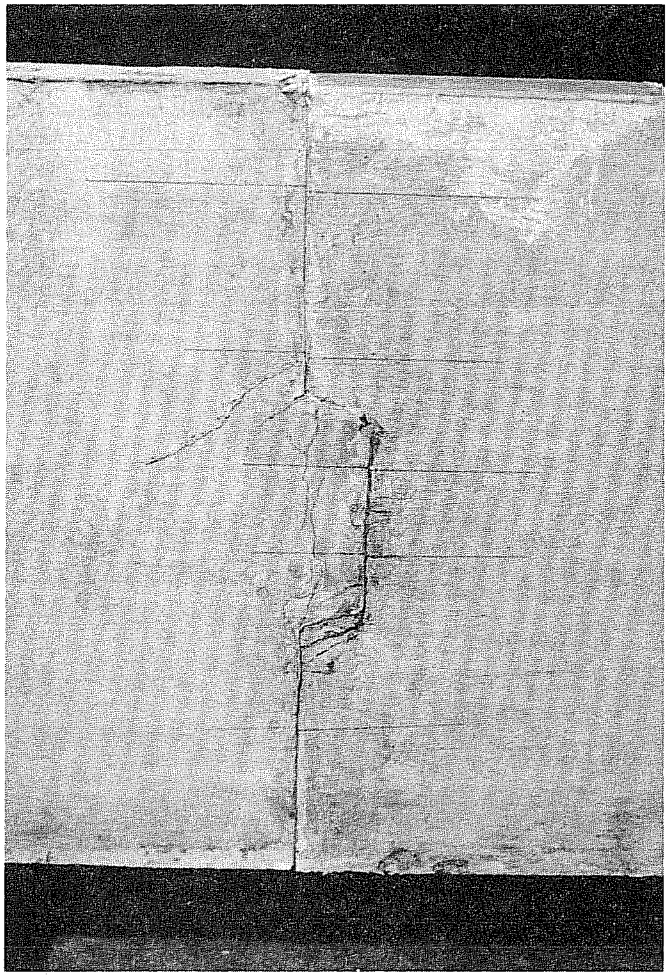


Fig. 3.9 Crack pattern at failure (single key joint w/o epoxying)

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.

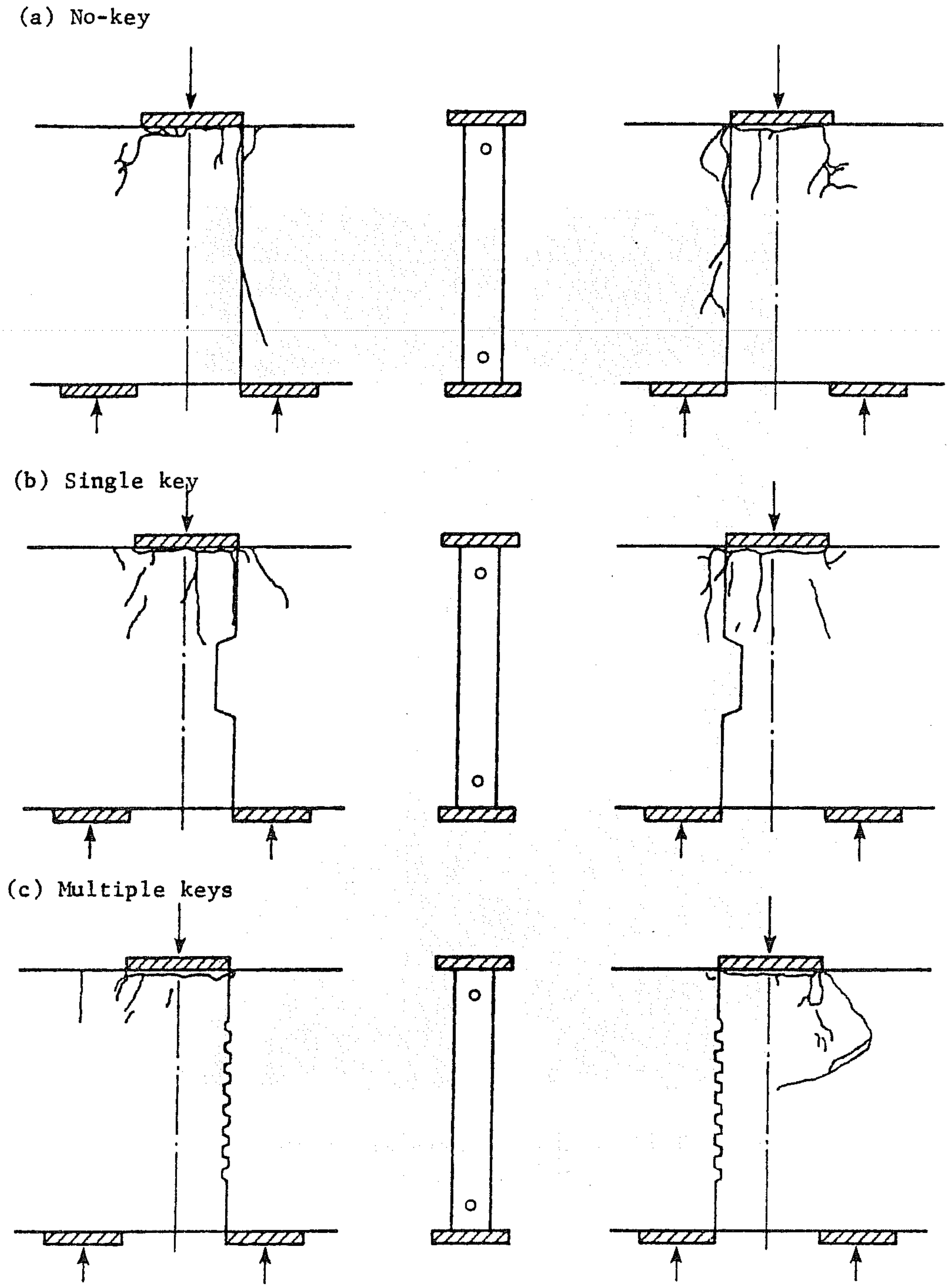


Fig. 3.12 Crack patterns at failure (specimens w/epoxying)

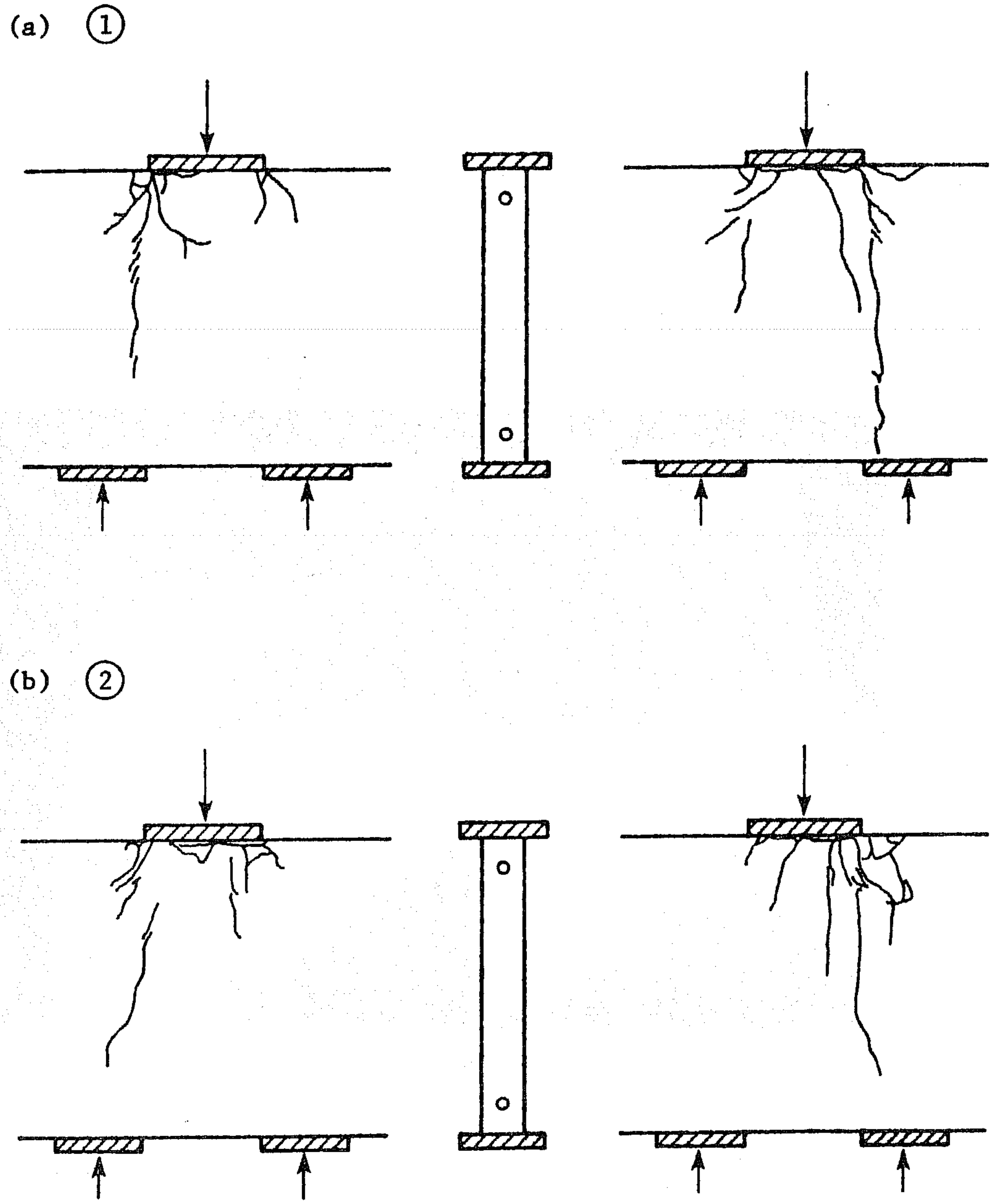


Fig. 3.14 Crack patterns at failure (no-joint)

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.

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

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

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

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

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

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

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.

$V_p = 0$

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

	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 μ = 1.0 are suggested for concrete placed against hardened concrete, while values of μ = 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, ϕ = 1.0 was used, since the material strength and specimen dimensions were accurately known.

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.

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 (μ= 0.61), while that for the dry no-key joint was 28 kips (μ = 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.

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 $9.6\sqrt{f'_c}$ ($>7.5\sqrt{f'_c}$).

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

$$= 0.96 \text{ k}$$

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

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

$$= 12.3 \text{ k}$$

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

$$= 12.7 \text{ k}$$

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

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

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 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 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

$$V_{n1} = mvb_w h$$

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

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

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

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.

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 Areas for further studies. 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 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

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

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.

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