THE USE OF EPOXY RESIN COMPOUNDS AS A SHEAR CONNECTOR IN COMPOSITE BEAM CONSTRUCTION

by

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ABSTRACT

This study was initiated to determine the critical shearing stresses which an epoxy bonded aggregate shear connector is capable of developing, and to determine the optimum physical and material characteristics of the connector when utilized in composite construction.

The critical shearing stresses were determined from tests conducted on six composite T-Beams which utilized an epoxy bonded aggregate shear connector. These beams were tested under two different temperature conditions and utilized different loading and supporting constraints.

The optimum physical and material characteristics of the epoxy bonded aggregate system were determined from pull-out specimens. The results of these tests determined the optimum epoxy bond line thickness, aggregate type, aggregate size, and aggregate coverage which is to be used in composite construction.

The results from these studies along with results obtained from previous studies have been utilized in developing design criteria and material specifications concerning the use of epoxy bonded aggregates as a shear connector in composite construction. These specifications were prepared by modifying the present A.A.S.H.O. specifications concerning composite beams.

CHAPTER 1

INTRODUCTION

1.1. Introduction

Among the many different types of bridges used in the highway systems of today, the composite bridge consisting of a rolled steel section for a stem and a reinforced-concrete slab as a roadway has become one of the more common. Due to its pleasing appearance and absence of structural members in the user's line of sight, the composite or deck bridge, as it is commonly called, has become increasingly popular in recent years.[1] In its early stage of development, such structures were built with the slab resting freely on the top flanges of the rolled section. Any interaction between the two structural elements was entirely dependent on the natural bond between the concrete and the steel. As the amount of traffic, as well as the load limits for automobiles and trucks, increased; it was found that the natural bond which existed between the concrete and steel was broken. The failure was attributed mainly to the increased frequency and magnitude of the repeated loads. It was, therefore, necessary to find a more efficient and reliable method by which to anchor the slab to the rolled section.

In the early 1930's, studies were begun in Switzerland on composite beams in which various types of mechanical shear connectors were used to transfer shear forces and to develop greater monolithic action. These studies were later supplemented by investigations in Europe and in the

United States.[2] It was from these studies that the shear connectors which are used in today's construction methods were developed. The mechanical shear connectors are welded to the top flanges of the rolled steel section and are in the form of studs, spirals, channels, and serpentine bars (see Figure 1). "These connectors act essentially as a dowel embedded in an elastic medium."[3]

Although such connectors are being used with good results, there are reasons for investigating new methods of joinery.

In normal construction procedures, beams and girders are usually positioned before the formwork for the slab has begun. Because high electrical currents are required for the application of mechanical connectors, the connectors are usually shop applied. In order to build the formwork for the concrete slab, the workmen are required to walk on the tops of the beams and as a result accidents and deaths have been caused by persons tripping over a connector. Concern for such safety hazards has recently resulted in the possible outlawing of the shop-applied shear connector in several of the major cities in the United States.

Due to the spacing of the shear connectors along the length of the beam, the shear forces are not transfered continuously from the slab to the stem. Such discontinuities are points of stress concentration. It is believed that these points of stress concentration contribute significantly to the cracking of the bridge decks which has become a major highway problem. This type of cracking is usually noted on bridges with high traffic flows.

These and other reasons have caused persons in the construction industry to search for other means of joining composite elements. It was

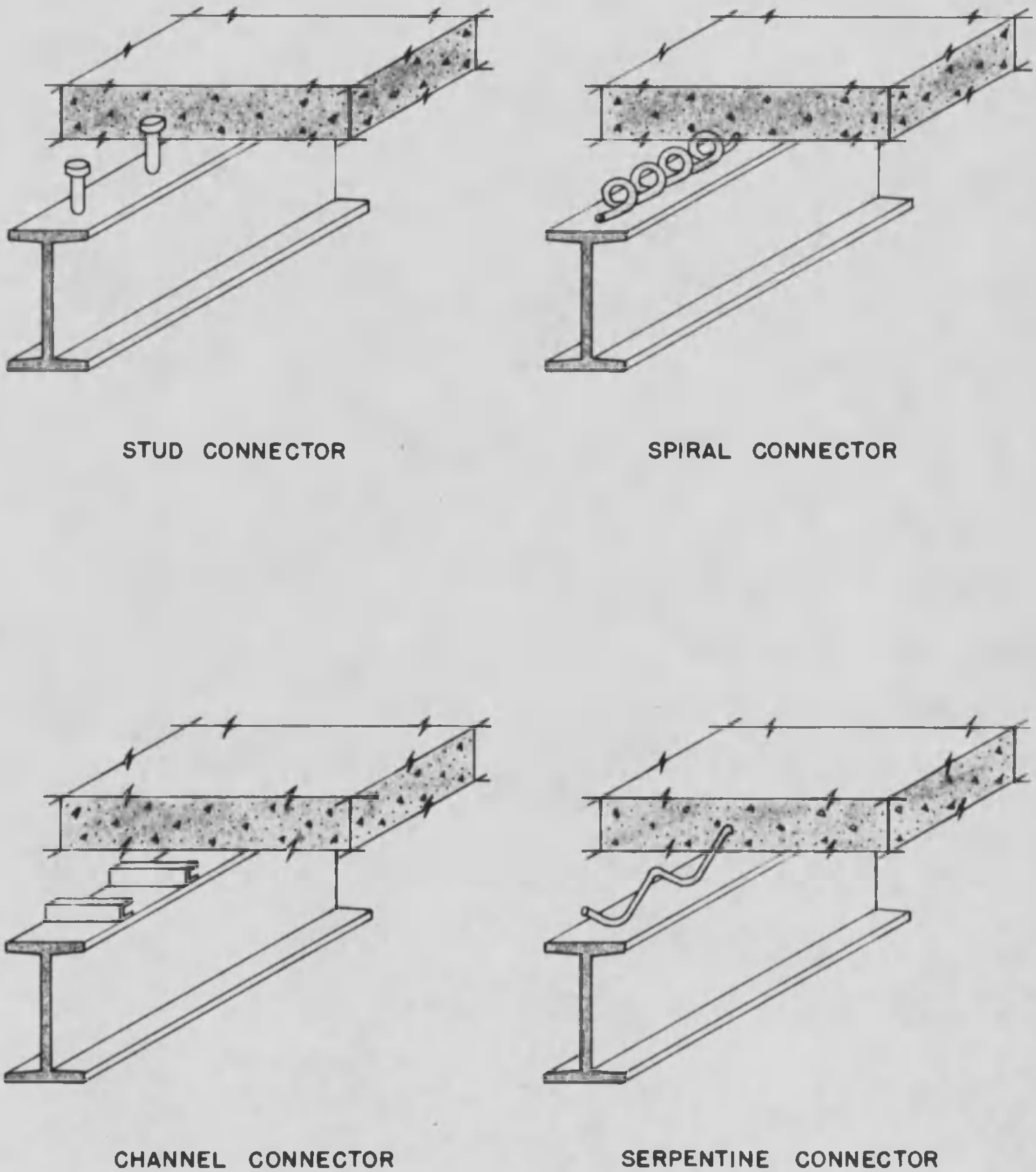


FIGURE 1 TYPICAL COMPOSITE SECTION WITH VARIOUS MECHANICAL SHEAR CONNECTORS

at this point that the question of the use of epoxy resins as a shear connector was raised. Introduction of epoxy resins into the field of structural adhesives has only recently been recognized as a feasible method for joining structural elements. Although their use has been confined mainly to laboratory tests and experiments, it has been shown that the use of epoxy resins in the construction field is highly feasible.

1.2. Purpose

Before epoxy resins can be used effectively and efficiently as shear connector in composite construction, it will be necessary to develop certain design procedures and specifications pertaining to its use. It is, therefore, the purpose of this thesis to suggest effective design procedures and construction techniques and to conclude with design specifications by modifying the present A.A.S.H.O. (American Association of State Highway Officials) specifications concerning composite beam design to include structural adhesives.

1.3. Scope

The purpose of the shear connector is to serve two functions: 1) to prevent relative movement between the slab and the stem, and 2) to transfer horizontal shear from the slab to the stem. The relative movement between the slab and the stem may be either horizontal or vertical. Therefore, the shear connector must be capable of anchoring the slab in both directions.

It has been shown that an epoxy system is capable of developing the necessary shear and bond strength for use in composite construction.[5] To determine the critical shearing stresses for design purposes six T-beams were designed and tested statically. Three of the T-beams were designed with 12I50 rolled steel sections 21 feet long as a stem, and the remaining three utilized 15I50 rolled steel sections 29 feet long. The concrete slab in all cases was 40" x 4" and extended the entire length of the beams. The width of the epoxy layer was designed in such a manner as to produce a shear-bond failure between the epoxy layer and the steel stem before initial yielding in the rolled section occured. One beam of each size was tested over continuous supports. The remaining two of each size were tested as simply supported beams; however, one beam of each pair was tested at room temperature and the other was tested at a temperature below zero degrees Fahrenheit.

In addition to the T-beam tests for determining the critical shearing stresses, another test was designed and performed to determine the optimum epoxy bond line thickness, and to evaluate the effects of various aggregates on the anchorage strength of the epoxy system.

The information obtained from these tests, along with the results from previous tests is used to develop the design criteria herein included.

1.4. Summary of Past Studies

Research studies on epoxy resins for use in the industrial construction fields began in Germany in the 1930's, but it has only been within the past few years that their use has been realized in the structural field. The construction industry has used epoxy resins in a variety of ways, concentrating mainly in the area of protective coatings and repair work for damaged concrete. The aircraft industry has used epoxy compounds as a structural adhesive and shear connector by utilizing the high shearing strength properties combined with the light weight and durability of these systems. Epoxy resins combined with glass fibers have also been used for storage tanks and booster cases for solid fuels in rockets.[4] Satisfactory results in the above areas indicated that epoxy systems might be used in other structural capacities. This led investigators at The University of Arizona and at other laboratories to investigate the possibility of using epoxy systems as a shear connector for composite T-beams.

In 1959 research began at the Civil Engineering Department, The University of Arizona, to determine whether or not an epoxy compound could serve as a suitable shear connector for composite T-beams. Many different types of epoxy systems were studied in order to establish a compound with physical properties necessary for such use. Upon the development of a suitable epoxy formulation, full scale tests on epoxy joined composite T-beams were conducted to show that an epoxy compound could be used for this purpose.[5] In 1963 further studies were initiated to determine an effective method for application and design of the epoxy shear connector. To aid in these studies tests were designed and performed for the purpose of 1) evaluating the effect of bond line thickness on bonding strength[6], 2) evaluating the effect of moisture penetration on the bonding strength of the epoxy[7], 3) evaluating the optimum proportions of the basic ingredients used in the initial epoxy formulation[8], 4) evaluating the effect of small variations in the ingredients[9], and 5) evaluating the effects of aging on the epoxy formulation.[10] In addition to the tests memtioned above, studies were made on various epoxy joined composite T-beams to evaluate the effects of dynamic and repeated loads.[11] One T-beam was tested at approximately zero degrees Fahrenheit to study cold temperature effects.[12] All of this above work led to the conclusion that the epoxy joined composite T-beam could be used in highway bridge construction if proper design criteria were developed.

In all of the above studies, the epoxy shear connector was applied simultaneously with the pouring of the concrete for the slab. Aggregate was placed in the fresh epoxy before placing the concrete to insure proper anchorage between the epoxy layer and the concrete slab.

In October, 1962, the Civil Engineering Department of Rensselaer Polytechnic Institute also published a report on their investigations in the field of epoxy resins. Their program was initiated to obtain more information on epoxy formulations, and to develop a specific epoxy formulation which could be used a shear connecting device in composite construction. Their investigations included comparative studies on the physical properties of various epoxy formulations. Having selected a formulation, further tests were conducted to obtain a complete set of physical properties of the formulation.

The next phase of their investigation included a series of tests designed to study the behavior of small size composite beams loaded statically to failure.

Finally, a cost comparison was made between the conventional method of composite construction, using welded studs as a shear connector and the proposed epoxy adhesive method.

Their studies indicated that it was possible to develop an epoxy formulation which would be suitable for application in the field of heavy construction, and that cost would not be a prohibitive factor.

In December, 1963, Rensselaer Polytechnic Institute published the final report on their investigations in the field of structural adhesives. It was basically an extension of their first report and included studies on creep, fatigue on glued joints, impact, strength gain or loss under various programmed temperature conditions, effect of changes in filler content, catalysts, flexibilizers, and deterioration characteristics of the materials. Also, the performance characteristics of their formulation as a shear connector was further studied by additional test on composite beams.

Recent studies conducted by the California State Highway Department have also shown that epoxy bonded aggregates could be used effectively as a shear connector in composite construction.

In all of the tests conducted by the California State Highway Department a precured epoxy system was utilized. The epoxy and aggregate were applied to the steel section and allowed to cure before placing the concrete.

Various tests were conducted to check the validity of the premise that epoxy bonded aggregates could serve as a suitable shear connector in composite construction. Push-out tests were conducted on samples of concrete that had been cast between two steel plates. Shear resistors in the form of epoxy bonded aggregates along with various combinations of simulated mechanical hold downs, were attached to the steel plates. Various sizes of aggregates were used to determine the optimum aggregate size to be used in future testing. Accelerated corrosion durability tests were also conducted on plate specimens with various epoxy bond line thicknesses. The specimens were placed in a 5% solution salt spray bath for 488 hours and then subjected to 12 cycles of $-10^{o}F$ to $+78^{o}F$ freeze-thaw cycles. The specimens were then returned to the salt spray bath for an additional 500 hours. Concrete was then cast to the plates and the specimens were then tested by the push-out method.

The final stage of their study included the testing of various prototype T-beams with epoxy bonded aggregate as the shear connector between the concrete slab and the rolled steel section.

Their results indicated that epoxy bonded aggregates could serve as a shear connector and that further study should be continued in this area.

CHAPTER 2

SHEAR CONNECTOR

2.1. Introduction

Recent discussions between California State Highway Department officials and research investigators of the Civil Engineering Department, The University of Arizona, has resulted in a unified method for the shop or field application of the epoxy shear connector.

In the studies conducted at The University of Arizona, the epoxy shear connector was applied simultaneously with the pouring of the concrete slab. That is, the epoxy was applied to the steel section, the aggregate placed in the fresh epoxy, and the concrete placed while the epoxy was still in the semi-liquid state.

Tests conducted by the California State Highway Department utilized a precured epoxy system on their composite beams. Except for the precuring, their system was identical to that developed by The University of Arizona investigators. The epoxy was applied to the steel section and aggregate placed randomly in the fresh epoxy. The epoxy was then allowed to cure for a sufficient period of time. When the epoxy had gained the required strength the concrete was placed.

Although very good results were obtained by the method used at The University of Arizona, it was decided that the precured system used by the California State Highway Department would be the most feasible method in field application. This decision was based on the following reasons: 1) Positioning of the deck and slab reinforcement over the steel section renders the job of placing the epoxy and aggregate simultaneously with the pouring of the concrete very difficult, if not almost impossible. With the precured system the epoxy could be applied in the fabrication shop or on the job site before the actual placing of the reinforcement steel or even before the steel sections are lifted into place. 2) Since proper shear and bond strength are an absolute necessity for the use of the epoxy system, quality control specimens could be taken at the same time as the pouring of the epoxy. If the precured epoxy system did not meet the necessary strength requirements, the beam could be recleaned and a new system applied before the bridge deck was poured. Using the other method, poor strength properties would not be detected until after the bridge deck had been poured or until actual failure of the connector had already occurred. Repair of such a failure would be very expensive and very difficult even under ideal conditions. 3) Proper curing of the epoxy system is highly dependent on the temperatures and humidity conditions. High temperatures reduce the pot life of the epoxy compound considerable, thus making field applications using the method utilized by The University of Arizona very difficult. Low temperatures do not allow the epoxy to gain its proper strength, thus causing failure at relatively low shearing stresses. Utilizing the precured epoxy system, combined with quality field control or shop application, the temperature and humidity could be reasonable controlled; thus insuring proper curing and easy handling of the epoxy system.

For these reasons the precured system has been chosen as the method of application in this study.

2.2. Description Of The Epoxy Compound Used For The Shear Connector

The epoxy system used in this study is the same formulation as that used in recent studies on composite T-beams at the Civil Engineering Department, The University of Arizona. The basic formulation ingredients were supplied by the Jones-Dabney Company. This formulation consists of two basic liquid components, the epoxy resin itself, and the hardening agent. Each basic component is chemically stable within itself for an indefinite period of time. These react to form a plastic solid only when these two components are thoroughly mixed. Other ingredients such as fillers and flexibilizers were also added to the basic formulation to impart the necessary physical properties to the epoxy system in both the liquid and solid states. A description of the formulation follows:

Formulation

(Two Component System)

Part A Resin Portion Constituents		Part B Converter Portion Constituents	
Epi-Rez 510	100 pbw	Epi-Cure 855	25 pbw
Alumina T-60	20 pbw	Epi-Cure 87	10 pbw
Asbestos 7-TF-1	20 pbw	Alumina T-60	21 pbw
		Asbestos 7-TF-1	21 pbw

Constants:

Weight 1 gallon, Resin Portion (lbs.) 11.77

Weight 1 gallon, Converter Portion (lbs.) 12.93

Weight 1 gallon, Total Formulation (lbs.) 12.16

Viscosity of Resin Portion at 1 RPM, 77°F (cps) 161,000

Viscosity of Resin Portion at 5 RPM, 77°F (cps) 100,000

Viscosity of Converter Portion at 1 RPM, 77°F (cps). . . . 3,300,000

Viscosity of Converter Portion at 5 RPM, 77°F (cps). . . . 1,280,000

Viscosity of Total Formulation at 1 RPM, 77°F (cps). 84,000

Viscosity of Total Formulation at 5 RPM, 77°F (cps). 35,200

Pot Life at 77°F, for one pint batch (minutes) 35

Gel time in a thin film at 77°F (hours) 2.0

Mixing Ratio:

1 quart resin portion to 1 pint converter portion

Mixing:

Blend two volumes of the resin portion with one volume of the converter portion; both portions having first been mixed individually. Thorough mixing can be accomplished by either manual or mechanical means, depending on the batch size.

Epi-Rez 510 is a standard epoxy resin. It is composed of commercial diglycidal ether of bisphenol A containing no added diluents, solvents or other contaminants. It has the ability to be transformed readily from the semi-liquid state to a tough, hard, thermoset plastic solid. This transformation may be brought about by several means; heat, a catalyst, a reactive hardener, or a combination of these. All of the above cause the resin molecules to join together in a complex cross-linked structure.

Epi-Cure 855 is a flexibilizer-hardener and is an aliphatic amido polyamine consisting of the reaction product of a long chain monobasic acid with polyamine. The use of Epi-Cure 855 in the formulation provides for two basic functions; partial catalyzation of the epoxy resin, and more important as a resinous modifier. Its effect on the cured system is to increase its ductility and provide for higher flexural strengths (decrease brittleness). This flexibilizer alone will react slowly with the epoxy resin; however, the addition of Epi-Cure 87 is necessary for a satisfactory cure.

Epi-Cure 87 is a hardening agent and is a modified aliphatic polyamine. Its function is to provide the chemical reaction causing the epoxy resin to be transformed into a plastic solid. It acts essentially as a reactive hardener in that it actually becomes part of the cured plastic molecule.

Alumina T-60 is a metalic filler. Its basic function is to impart more desirable handling characteristics to the uncured epoxy system and to improve strength properties of the cured system. The viscosity of the epoxy system is increased thus allowing for easier handling and application. Besides significantly affecting tensile, bond, and shear strengths; such fillers are also added to reduce costs, reduce shrinkage, lower the coefficient of thermal expansion, and increase the pot life of the epoxy resin system.

Asbestos 7-TF-1 is a thixotropic filler. Its addition into the formulation is to prevent the Alumina T-60 from settling out before initial set in the epoxy resin occurs. In addition, it also imparts some of the same property changes as described in the above paragraph on Alumina T-60.

This adhesive is designed for bonding concrete to steel. The concrete can be either old concrete which has been cleaned or freshly poured concrete. This adhesive cures well in the presence of moisture. The steel should be free from mill scale, oil, and rust. For ease of application, tools such as trowels are best suited.

Material Sources:

Epi-Res 510	Jones-Dabney Company
Epi-Cure 855.	Jones-Dabney Company
Epi-Cure 87	Jones-Dabney Company
Alumina T-60	Aluminum Company of America
Asbestos 7-TF-1	Johns Manville Company

Caution:

The use of rubber gloves, goggles, and protective clothing is necessary in areas where Epi-Rez 510 and converters are handled and used. Areas of the body which come in contact with the resin and/or converters should be cleaned with denatured alcohol, and then well scrubbed with soap and water.

Comments:

The above description is suited for the compound when used as a two-component mix. However, it was found in the laboratory that a more reliable mix could be achieved by using a three component system.

This three component system is described below. The constants would have to be adjusted accordingly:

Formulation

(Three Component System)

Part A Constituents	pbw	Part B Constituents	pbw	Part C Constituents	pbw
Epi-Rez 510	100	Epi-Cure 855	25	Alumina T-60	41
		Epi-Cure 87	10	Asbestos 7-TF-1	41

Parts A and B are thoroughly mixed by mechanical or manual means. After this mix is completed, Part C is added and thoroughly stirred into the mixture. By this method a better chemical reaction is assured, particularly on days with temperatures from $60^{\circ}F$ to $70^{\circ}F$. It is recommended that a three component system be used for this type of work regardless of the temperature.

2.3. Physical Properties of the Epoxy Formulation

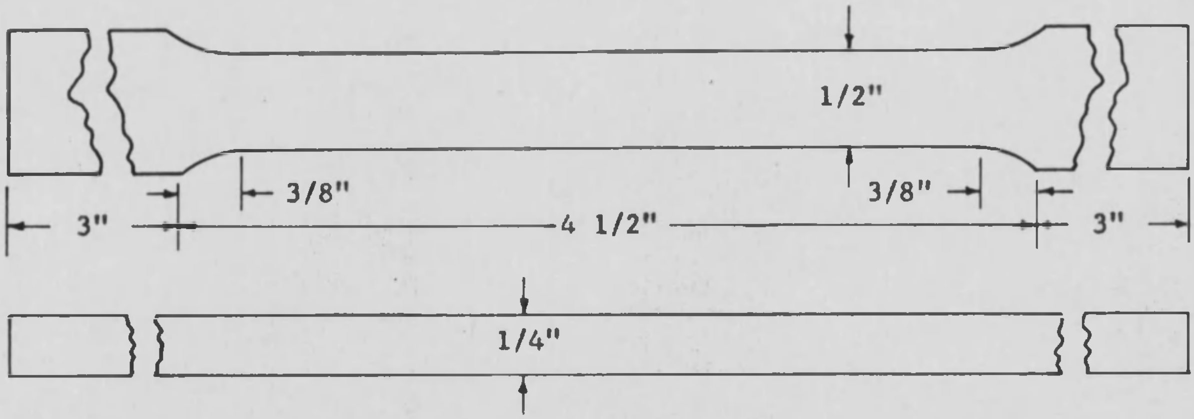
The physical properties described herein are those which have been determined at the Civil Engineering Department, The University of Arizona, in their recent investigations on the epoxy formulation.

2.3.1. Tensile Adhesive and Shear Adhesive Tests

Ultimate tests for the tensile adhesive and shear adhesive properties were conducted similar to ASTM specifications D 1002-53T and D 1062-51. These tests were conducted to evaluate the ability of the epoxy formulation to develop tensile and shear strengths when bonded to steel. A minimum of 10 tests were conducted in determining a single average strength value. The tensile adhesive test specimens consisted of two circular steel cylinders with cross-sectional areas of one square inch. These cylinders were bonded together with a thin epoxy resin film. A special jig was constructed to hold the specimens a distance of 0.014 inches apart and to assure vertical alignment of the cylinders. (See Figure 2.) After the specimens had been properly cured, they were tested in a direct tension as shown in Figure 3. The shear adhesive specimens consisted of two lapped steel straps 0.060 inches thick and one inch wide, with a contact area of 0.50 square inch. The bond-line thickness was maintained at 0.014 inches. (See Figure 4.) After proper curing the specimens were tested as shown in Figure 5. For each test the ultimate strength and type of failure were noted. On the basis of past tests on the epoxy formulation values of approximately 5000 psi and 2000 psi were obtained for the tensile adhesive and lap-shear adhesive strengths respectively. It was also noted that when proper strengths were obtained, the failure occurred in the epoxy film rather than as a bond failure between the steel and epoxy.

2.3.2. Material Tensile and Compressive Tests

Tensile and compressive tests previously performed on the epoxy formulation indicate a range of values for the modulus of elasticity of 500,000 to 600,000 psi for tension and 250,000 to 350,000 psi for compression. The tests were performed according to ASTM specifications for plastics and conducted at a room temperature of $72^{\circ} \pm 5^{\circ}F$. The tension specimens were formed in silicone rubber molds with the following dimensions:



The tests for tension were conducted with an Instron testing machine with a constant cross-head speed of 0.5 inches per minute (see Figure 6). Stress-strain curves were plotted simultaneously with the testing of each specimen. A typical stress-strain curve for the epoxy formulation in tension is shown in Figure 7. Compression tests were conducted on molded epoxy cylinders one inch in diameter and two inches long. The compressive modulus of elasticity was measured by SR-4 strain gages mounted on the cylinders. (See Figure 8.) A typical stress-strain curve for the compressive cylinder is shown in Figure 9.

2.3.3. Poisson's Ratio

A determination of Poisson's Ratio was also conducted on a tensile specimen and on a compression cylinder specimen. Results indicate that a value of 0.40 is very reasonable.

2.3.4. Aging Effects

In recent studies conducted at the Civil Engineering Department, The University of Arizona, attempts were made to determine the aging effects on the strength of the epoxy resin formulation.[10] The

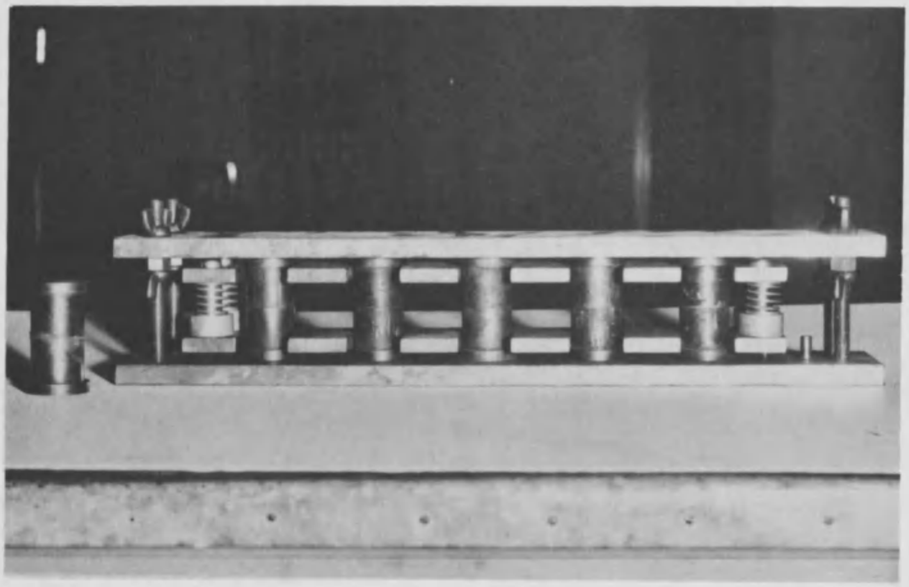


FIGURE 2 SPECIAL JIG FOR HOLDING ADHESIVE TENSILE SPECIMENS

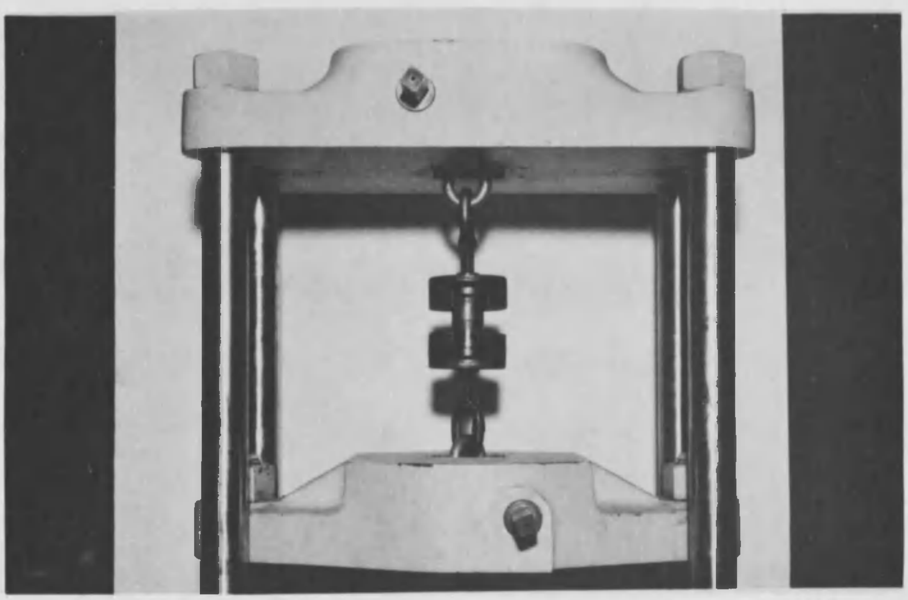


FIGURE 3 TESTING OF AN ADHESIVE TENSILE SPECIMEN

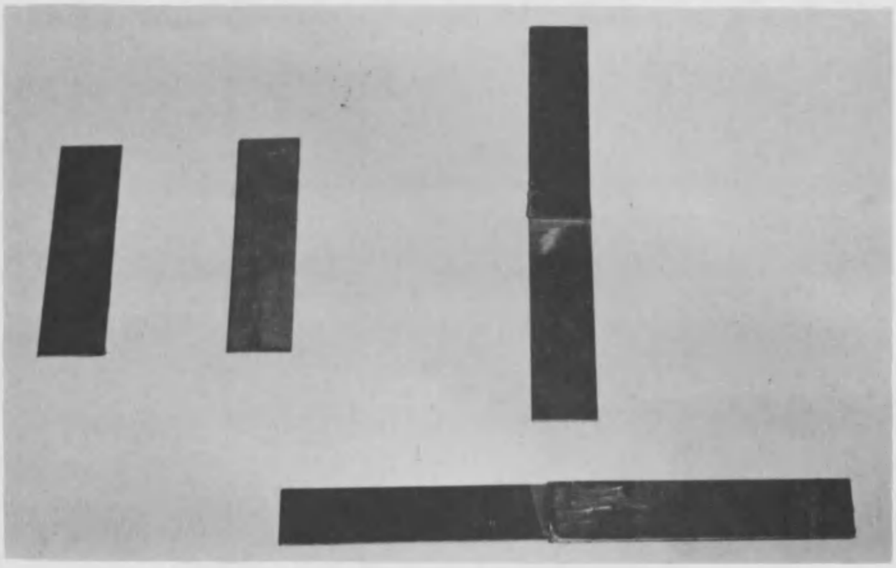


FIGURE 4 LAP TENSILE SHEAR SPECIMENS

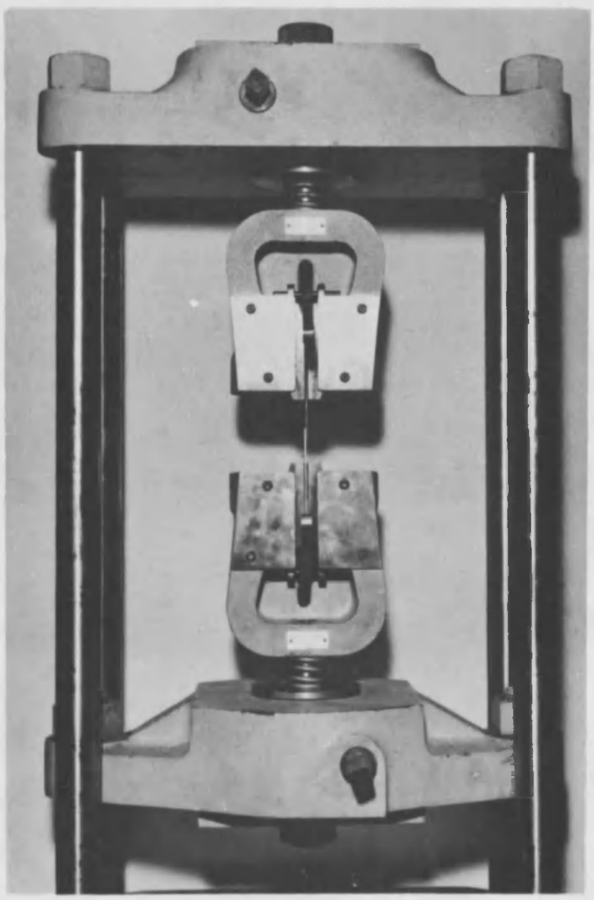


FIGURE 5 TESTING OF A LAP SHEAR SPECIMEN

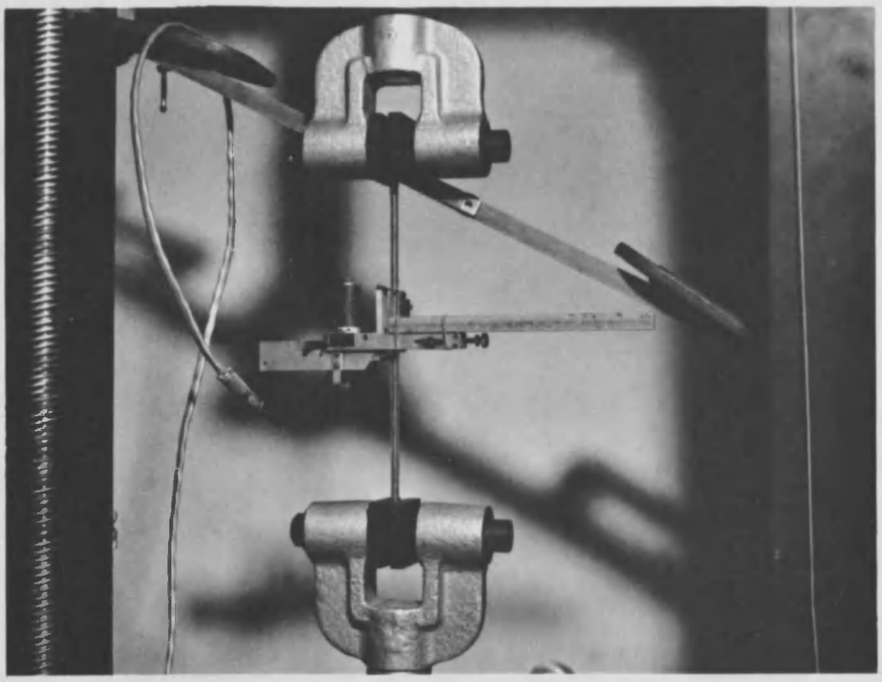


FIGURE 6 TESTING OF A TENSILE SPECIMEN

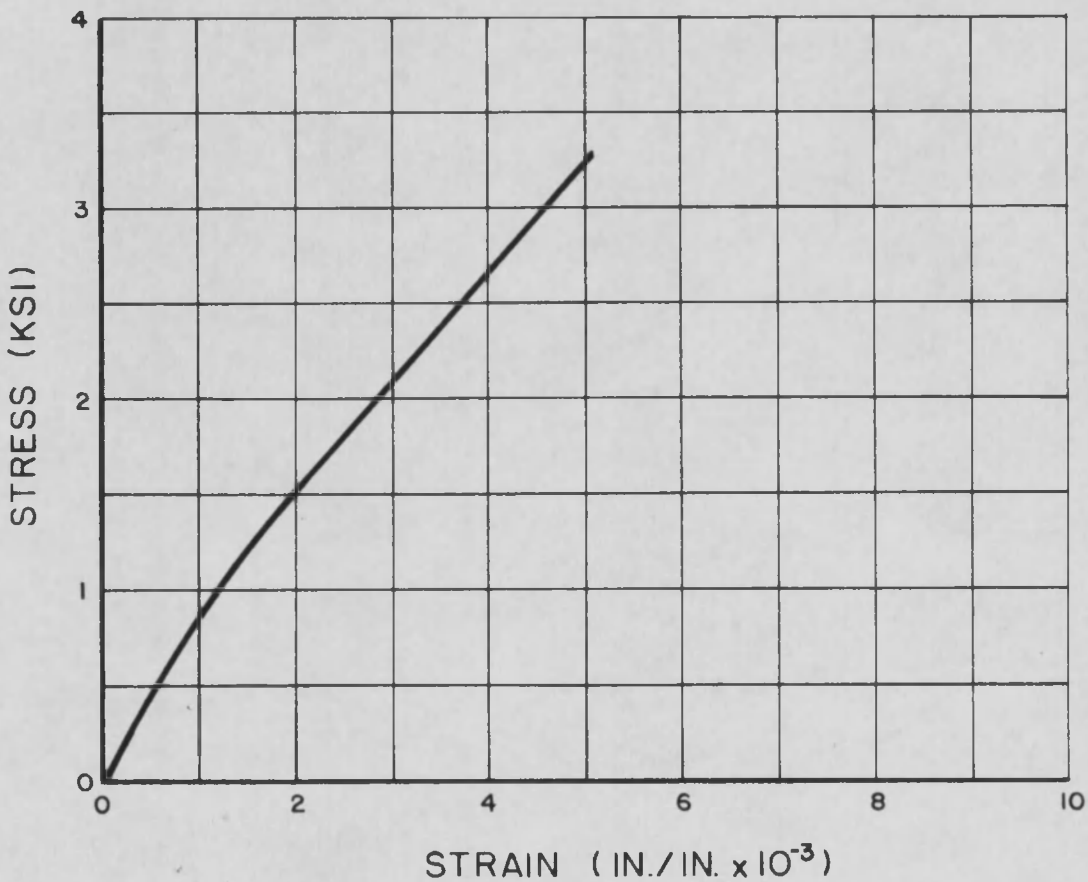


FIGURE 7 TYPICAL STRESS-STRAIN CURVE FOR EPOXY RESIN TENSILE SPECIMEN

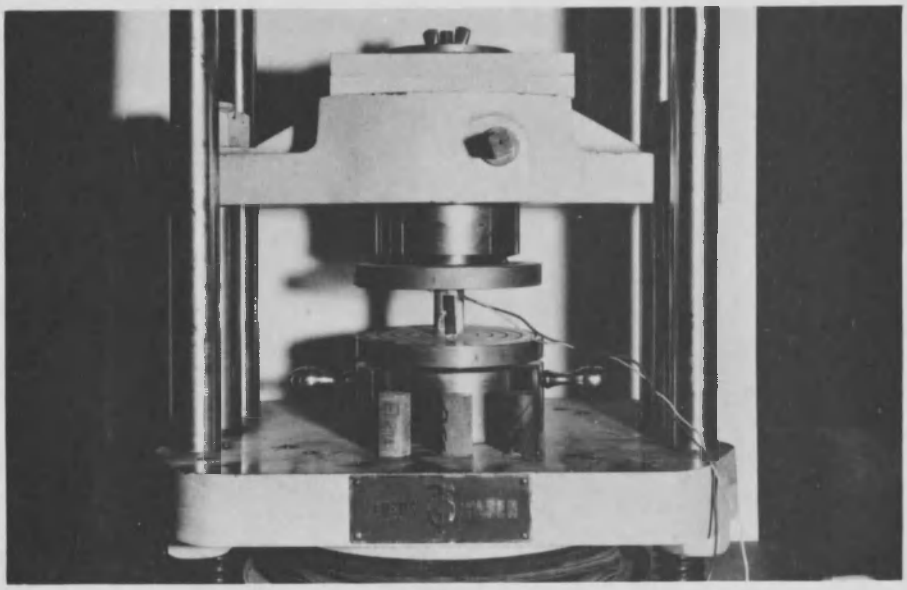


FIGURE 8 TESTING OF A COMPRESSION CYLINDER

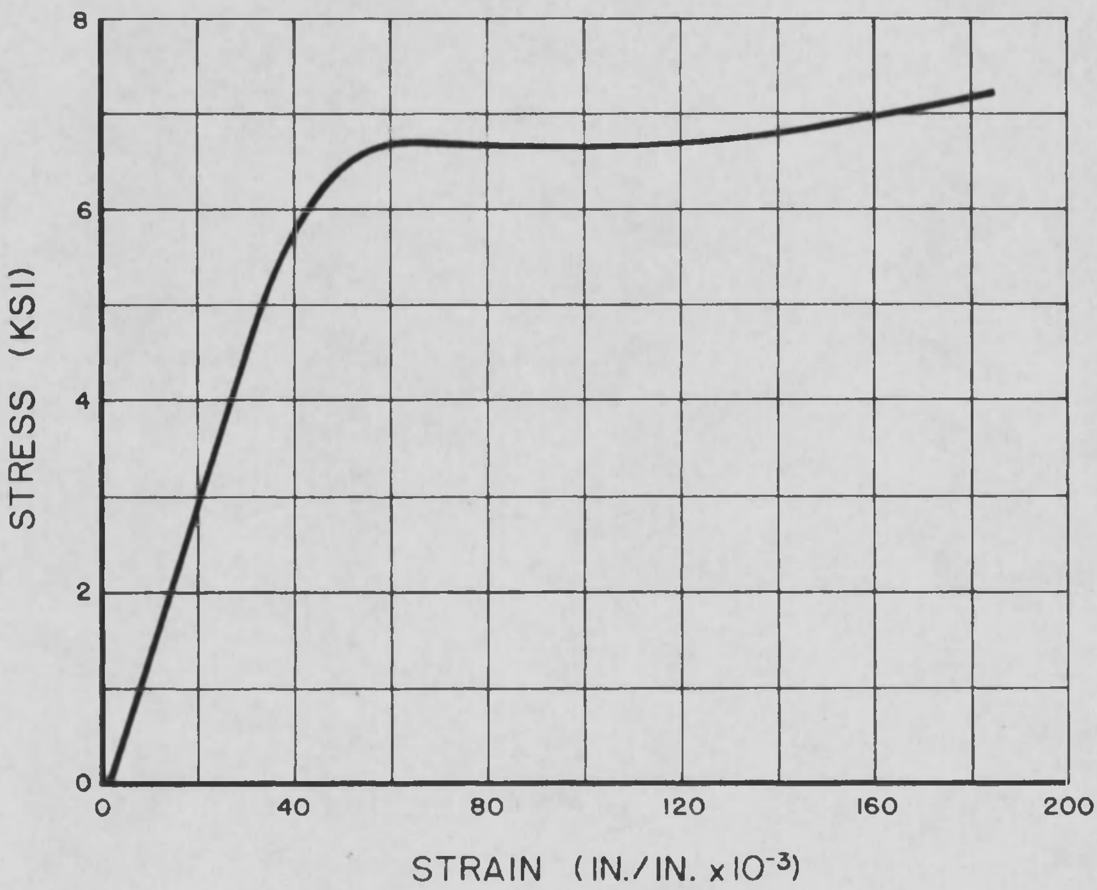


FIGURE 9 TYPICAL STRESS-STRAIN CURVE FOR EPOXY RESIN COMPRESSIVE CYLINDER

specimens were subjected to various climatic environments and then were tested.

Shrinkage stress as determined from the various tests indicated values from 50 to 75 psi assuming the stresses were constant throughout the specimen. However, these stresses were relatively low when compared to its tensile adhesive strength of approximately 5000 psi or even compared to its material tensile strength of about 3300 psi.

Aging was only apparent from the reduction in strength of the tensile adhesive specimens that were subjected to 100% humidity. In the report to the Arizona Highway Department of 24 January 1963, "The Use Of Epoxy Resins In Reinforced Concrete-Static Load Tests," it was noted that the thickness of the epoxy layer on the bonding surface affected the deterioration of the epoxy-to-steel bond when concrete was poured on the same liquid epoxy system. The results of the tests showed that moisture penetrates approximately 1/16 inch of the epoxy layer. For small layers the moisture penetrated to the steel which caused rusting of the steel and, in turn, decreased the strength of the bond. The thicker layer, as reported, prevented the moisture from contacting the bonding surfaces. This moisture penetration did not affect the strength of the epoxy.

2.4. Aggregate Pull-Out Tests

2.4.1. Introduction

The ability of the epoxy resin formulation to develop adequate tensile adhesive and lap-shear adhesive strengths for a shear connector has been shown in results obtained at the University of Arizona in their studies on the epoxy formulation. However, introduction of the precured system utilized by the California State Highway Department has brought about the need for further studies of materials and application techniques.

The epoxy resin provides the bond between the epoxy layer and the steel. The aggregate in turn provides the anchorage between the concrete slab and the steel stem; the concrete being poured after the epoxy resin has completely cured. Since there is no bond between the concrete and epoxy, the anchorage will be highly dependent on the physical and material characteristics of the aggregate which is to be used in the connector.

The tests described herein were designed to determine the optimum size, type, and percentage of coverage of the aggregate, and to determine the optimum epoxy bond line thickness, which is to be utilized in the design of an epoxy-bonded aggregate shear connector.

2.4.2. Grouping of Test Specimens

The test specimens were divided into fourteen basic aggregate groups. These groups consisted of thirteen aggregate types and gradations provided by the California State Highway Department, and one aggregate type provided by San Xavier Rock and Sand Company, Tucson, Arizona. The physical and material properties of these aggregates are shown in Table 1.

Twelve Specimens of each aggregate group were prepared. Of these twelve, six specimens were prepared with an epoxy bond line thickness of 3/8 inch, and the remaining six specimens with an epoxy bond line thickness of 1/4 inch. Each group of six specimens were in turn divided

TABLE I

PHYSICAL AND MATERIAL PROPERTIES OF AGGREGATES

AGGREGATE TYPE	GRADATION	SOUNDNESS Na_2SO_4	L.A. RATTLER		S.G.	% ABS.	PARTICLE DESCRIPTION
			100 R	500 R			
GRANITE ROCK	1-1/2"—1"	2.3	6.0	26.0	2.89	0.8	CRUSHED
	1"—3/4"	2.3	6.0	26.0	2.89	0.8	
	3/4"—3/8"	2.3	6.0	26.0	2.89	1.4	
BEAR RIVER ROCK	1-1/2"—1"	1.7	8.4	36.0	2.61	0.8	ROUNDED VERY SMOOTH
	1"—3/4"	1.7	8.4	36.0	2.61	0.8	
	3/4"—3/8"	1.4	8.4	36.0	2.61	1.0	
PLEASANTON	1-1/2"—1"	2.7	5.0	24.0	2.70	1.3	ROUNDED SOME CRUSHED
	1"—3/4"	2.7	5.0	24.0	2.70	1.3	
	3/4"—3/8"	2.7	5.0	24.0	2.70	1.5	
MISSION VALLEY CONSOLIDATED	1-1/2"—1"	5.0	4.0	18.0	2.62	1.2	PARTIALLY CRUSHED
	1"—3/4"	5.0	4.0	18.0	2.62	1.2	
	3/4"—3/8"	5.0	4.0	18.0	2.62	1.2	
BEAR RIVER QUARTZ	3/4"—3/8"						CRUSHED
SAN XAVIER	1-1/2"—1"				2.58	1.59	CRUSHED

into two groups of three specimens each, one group utilizing 50 percent aggregate coverage and the other group utilizing 100 percent aggregate coverage. Thus, four separate sets of conditions were studied for each aggregate group. Three specimens of each condition were tested to obtain an average strength value and to observe and record the type of failure.

2.4.3. Preparation of Test Specimens

Steel disks four inches in diameter were carefully machined to a thickness of one inch. The top surfaces of which were surface ground to a highly smooth finish. The backsides of the disks were drilled and tapped at the center with a hole 3/4 inch deep and 7/8 inch in diameter; this threaded hole was utilized for testing purposes only. Prior to application of the epoxy resin the bonding surfaces were cleaned with methylethyl ketone. A paper form was then placed around the disks to insure the proper epoxy bond line thickness. The epoxy formulation was then applied to the bonding surfaces of the disks and the aggregate was immediately hand-placed into the fresh epoxy. The aggregate was pushed down slightly into the epoxy to insure proper bonding to the aggregate. These disks, with the epoxy and aggregate in place, were then allowed to cure at room temperature for seven days. After curing, concrete cylinders six inches in diameter and six inches high were cast over the epoxy bonded aggregate specimens. The completed specimens were then cured for fourteen days at $72^{\circ} \pm 5^{\circ}F$ and at 100 percent humidity. Figures 10 and 11 show the disks with the epoxy bonded aggregate in place before the concrete cylinders had been cast.

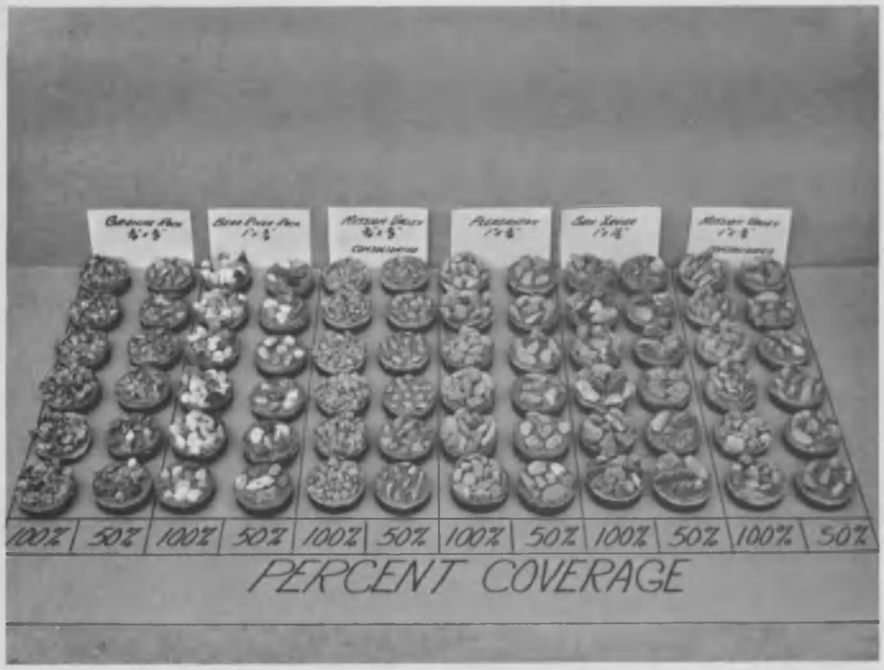


FIGURE 10 AGGREGATE PULL-OUT SPECIMENS BEFORE CONCRETE CYLINDERS WERE CAST

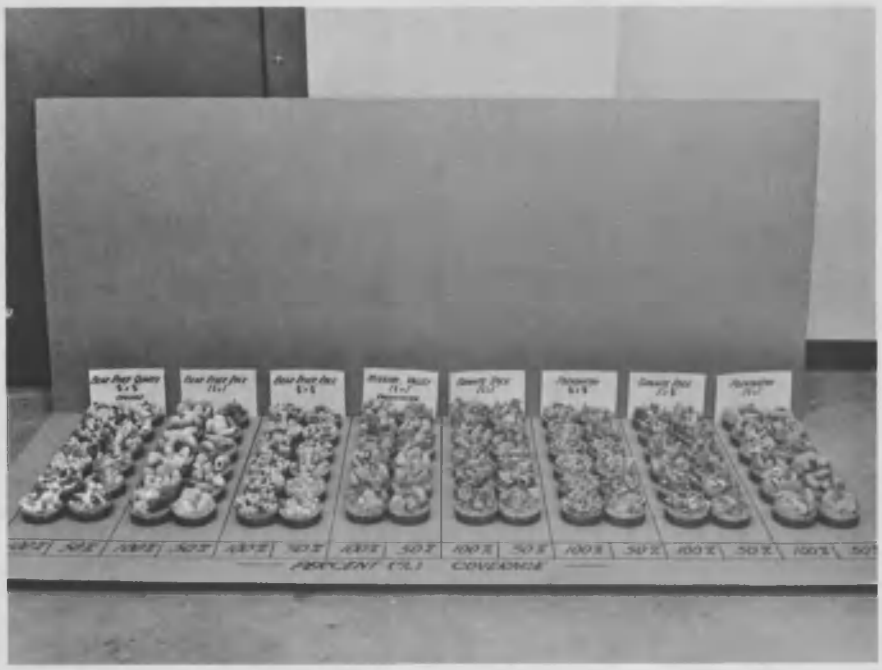


FIGURE 11 AGGREGATE PULL-OUT SPECIMENS BEFORE CONCRETE CYLINDERS WERE CAST

2.4.4. Testing of Aggregate Specimens

The specimens were tested in direct tension with a Tinius-Olsen 60 kip Universal Hydraulic Testing Machine as shown in Figure 12. The ultimate load and the type of failure were recorded for all tests. Figures 13 and 14 show typical disks after they had been tested.

2.4.5. Results of Aggregate Pull-Out Specimens

The test results of the aggregate pull-out specimens are shown in Table 2 and in Figures 15 through 18. Table 2 shows the average ultimate load in each of the specimen groups tested. Figures 15 through 18 give a graphical representation of the ultimate loads obtained with the various aggregate gradations, aggregate coverages, and epoxy bond line thicknesses.

As mentioned previously, the ability of the epoxy bonded aggregate system to provide vertical anchorage is highly dependent on the physical and material properties of the aggregate used in the connector. It was noted during the testing procedure that the type of failure produced within the aggregate gave a good indication of the aggregate's ability to provide anchorage. As can be seen in Table 2, the aggregate failure has been divided into two types; that is, the percent broken and the percent pulled. The percent of aggregate broken indicates the amount of aggregate in the specimen which was fractured or broken during the test. The percent of aggregate pulled indicates the amount of aggregate in the specimen which was pulled free of the concrete and/or the epoxy. Interpretation of these percentages will be discussed in Section 2.4.8.

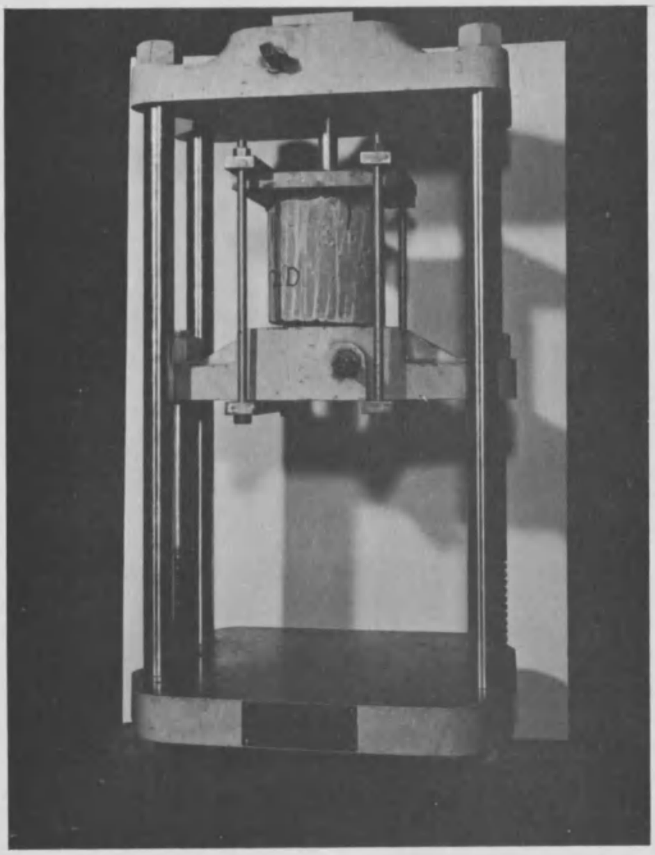


FIGURE 12 TEST SET-UP FOR AGGREGATE PULL-OUT SPECIMENS

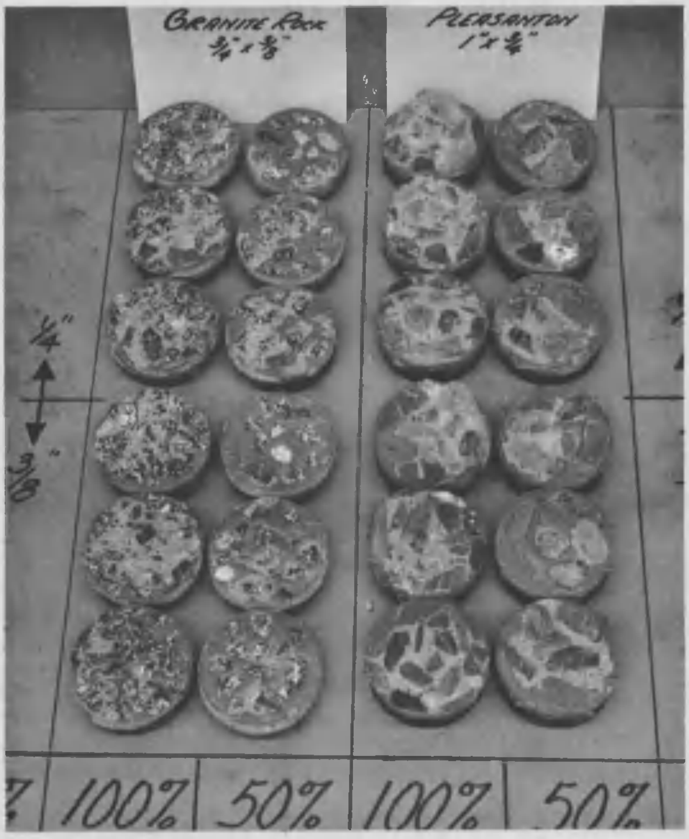


FIGURE 13 AGGREGATE PULL-OUT SPECIMENS
AFTER FAILURE

FIGURE 14 AGGREGATE PULL-OUT SPECIMENS AFTER FAILURE

2.4.6. Optimum Epoxy Bond Line Thickness and Percent Aggregate Coverage

As can be seen in Table 2 and in Figures 15 through 18, the specimens which utilized an epoxy thickness of 3/8 inch and 100 percent aggregate coverage produced the highest ultimate loads in virtually all of the cases tested. It was noted during the testing procedures that the specimens which utilized the 3/8 inch epoxy thickness provided greater anchorage or bonding between the aggregate and epoxy. Specimens with this epoxy thickness exhibited less tendency for the aggregate to be pulled free from the epoxy layer. The specimens which utilized 100 percent aggregate coverage also produced higher ultimate loads. This was a direct result of the increased amount of bonding area provided.

It should also be noted by examining Table 2 that the percentage of aggregate broken at failure was generally higher in the 50 percent coverage specimens. This indicates that the ultimate bond is dependent on the aggregate strength and that a larger amount of aggregate would produce a higher ultimate load.

2.4.7. Optimum Aggregate Size

Load-gradation curves for the granite and Bear River aggregates clearly indicate that the larger-sized aggregates produced the highest ultimate load.

The load-gradation curves for the Pleasanton aggregate indicate that higher ultimate loads were obtained for the 3/8 to 3/4 inch and 1 to 1½ inch gradations, while the 3/4 to 1 inch gradation produced the lowest ultimate strengths. These results are not consistant with those obtained from the granite and Bear River aggregates; however, it is thought that non-uniformity in the particle size and shape within the various gradations affected the outcome of the results. With the exception of one case, the 1-1½ inch gradation provided higher ultimate loads than did the 3/8-3/4 inch gradation.

As can be seen from the slight downward slope of the load-gradation curves for the Mission Valley aggregate the smaller aggregate sizes generally provided slightly higher ultimate loads. The percent of difference between the maximum and minimum ultimate loads in all cases was less then 10 percent, and it can be concluded that the aggregate gradations within this group had little or no effect on the anchorage ability of the epoxy bonded aggregate system.

On the basis of these test results it can be concluded that an aggregate gradation of 1-1½ inch will provide the best anchorage of the epoxy layer to the concrete.

2.4.8. Effect of Aggregate Properties on Anchorage

Since there is no bond produced between the concrete and epoxy, it can be seen that the physical as well as the material properties of the aggregate will have a large effect on the ability of shear connector to provide anchorage.

The ability of the aggregate to provide anchorage was dependent on the strength of the aggregate and equally dependent on the particle size, shape, and surface characteristics.

The surface characteristics of the aggregate utilized in the shear connector generally determined the type of failure produced in the aggregate. As can be seen in Table 2 the failure in the crushed aggregates (Granite Rock, Bear River Quartz, San Xavier) was produced almost entirely by the breaking or fracturing of the aggregate particles. This type of failure indicates that good aggregate bonding is produced with the concrete and epoxy. Conversely, the type of failure produced in the specimens with rounded and smooth surfaces (Bear River Rock, Pleasanton) usually consisted of a higher percentage of aggregate particles being pulled free of the concrete and/or epoxy. This type of failure indicated a loss of aggregate bonding between concrete and/or epoxy.

Sodium sulfate (Na_2SO_4) soundness tests conducted on the aggregates give an indication of the durability and strength of the aggregates. As can be seen from the test results in Table 2 the aggregates with the highest durability or strength do not necessarily provide the highest ultimate load. However, it can be realized that aggregates with extremely low durability and strength would not function properly when utilized in the epoxy shear connector.

The optimum aggregate type utilized in the shear connector is, therefore, a combination of good geometrical and material properties.

On the basis of the test results obtained by testing the various aggregate types, it can be concluded that aggregates with the following properties will provide the maximum anchorage and highest ultimate loads: (1) high durability and soundness (2) crushed aggregates containing particles with very rough surfaces (3) particles which are angular in shape.

2.4.9. Summary of Pull-Out Test Results

Results obtained from the aggregate pull-out specimens indicate that the best anchorage and highest ultimate strengths were produced by the following conditions: (1) utilization of the 100 percent aggregate coverage, (2) an aggregate gradation of 1-1½ inch, (3) aggregates with good strength properties, (4) an epoxy bond line thickness of 3/8 inch. Material properties and specifications are further discussed in Chapter 4.

TABLE 2

ULTIMATE STRENGTH AND TYPE OF FAILURE FOR PULL-OUT SPECIMENS

% COVERAGE & EPOXY THICKNESS		AGGREGATE GRADATION 3/4"—3/8" AVG. LOAD (lbs)	AGGR. FAILURE % BROKEN	AGGR. FAILURE % PULLED	1"—3/4" AVG. LOAD (lbs)	AGGR. FAILURE % BROKEN	AGGR. FAILURE % PULLED	1-1/2"—1" AVG. LOAD (lbs)	AGGR. FAILURE % BROKEN	AGGR. FAILURE % PULLED
GRANITE ROCK										
100%	3/8"	5700	100	0	6670	100	0	6837	90	10
100%	1/4"	5077	90	10	6063	100	0	6263	80	20
50%	3/8"	3225	100	0	4507	100	0	5477	100	0
50%	1/4"	2793	95	5	3717	100	0	4590	100	0
BEAR RIVER ROCK										
100%	3/8"	4507	50	50	5703	55	45	5673	50	50
100%	1/4"	4367	10	90	5120	50	50	4860	60	40
50%	3/8"	3233	70	30	3353	50	50	3520	60	40
50%	1/4"	2743	60	40	3153	50	50	3196	30	70
PLEASANTON										
100%	3/8"	7600	50	50	5533	70	30	6633	60	40
100%	1/4"	5287	50	50	4783	25	75	5817	70	30
50%	3/8"	3837	95	5	2950	75	25	5625	80	20
50%	1/4"	3523	95	5	2627	70	30	3913	90	10

TABLE 2 CONT.

% COVERAGE & EPOXY THICKNESS		AGGREGATE GRADATION								
		3/4"–3/8"			1"–3/4"			1-1/2"–1"		
		AVG. LOAD (lbs)	AGGR. FAILURE		AVG. LOAD (lbs)	AGGR. FAILURE		AVG. LOAD (lbs)	AGGR. FAILURE	
			% BROKEN	% PULLED		% BROKEN	% PULLED		% BROKEN	% PULLED
MISSION VALLEY CONSOLIDATED										
100%	3/8"	5870	15	85	5613	20	80	5473	75	25
100%	1/4"	5120	10	90	5590	15	85	5367	70	30
50%	3/8"	3337	40	60	3010	50	50	3017	60	40
50%	1/4"	3020	40	60	2983	100	0	2787	85	15
BEAR RIVER QUARTZ										
100%	3/8"	3723	50	50						
100%	1/4"	4240	85	15						
50%	3/8"	2813	95	5						
50%	1/4"	3063	95	5						
SAN XAVIER										
100%	3/8"							6143	100	0
100%	1/4"							5623	90	10
50%	3/8"							4293	100	0
50%	1/4"							3507	100	0

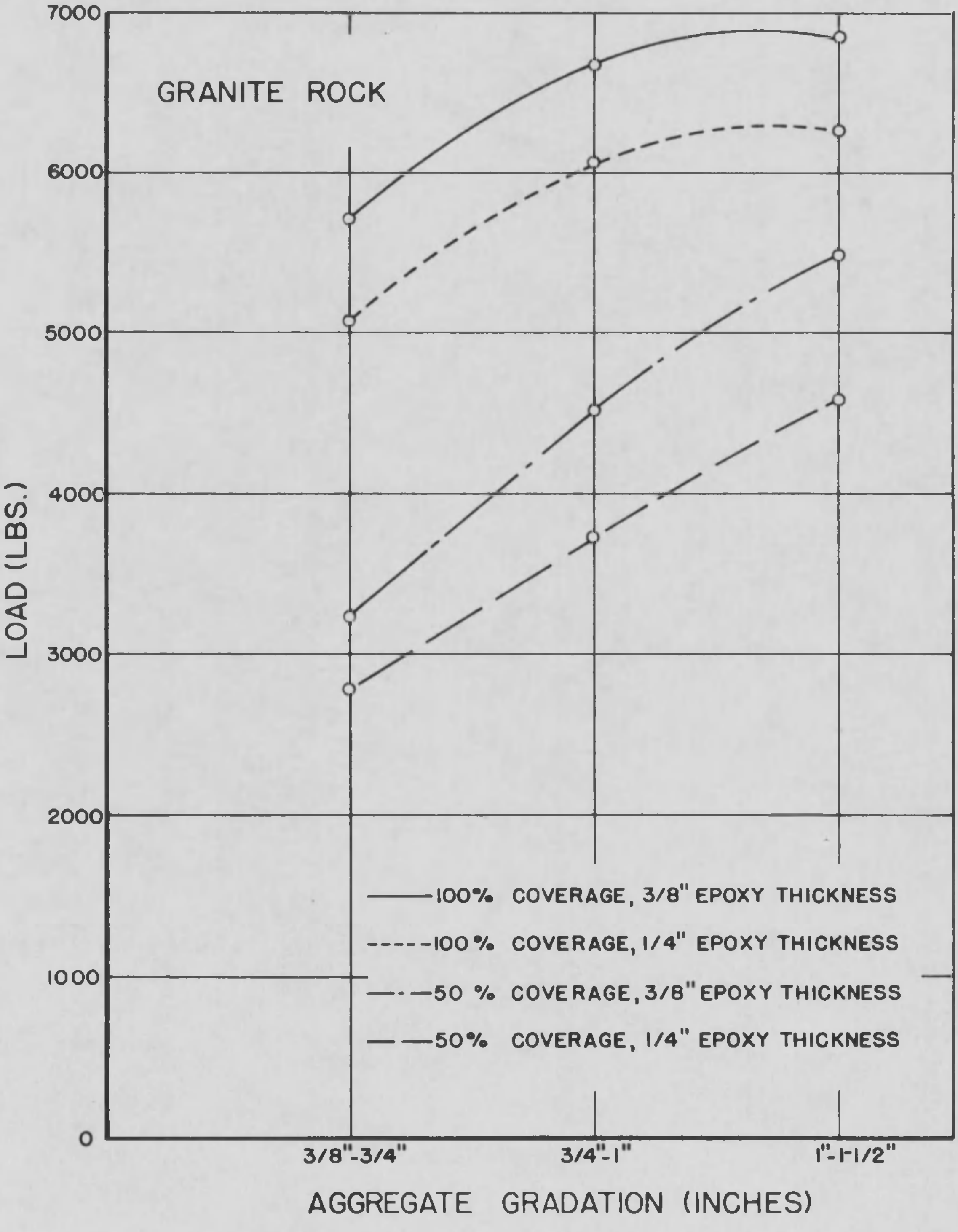


FIGURE 15 LOAD-GRADATION CURVES FOR GRANITE ROCK

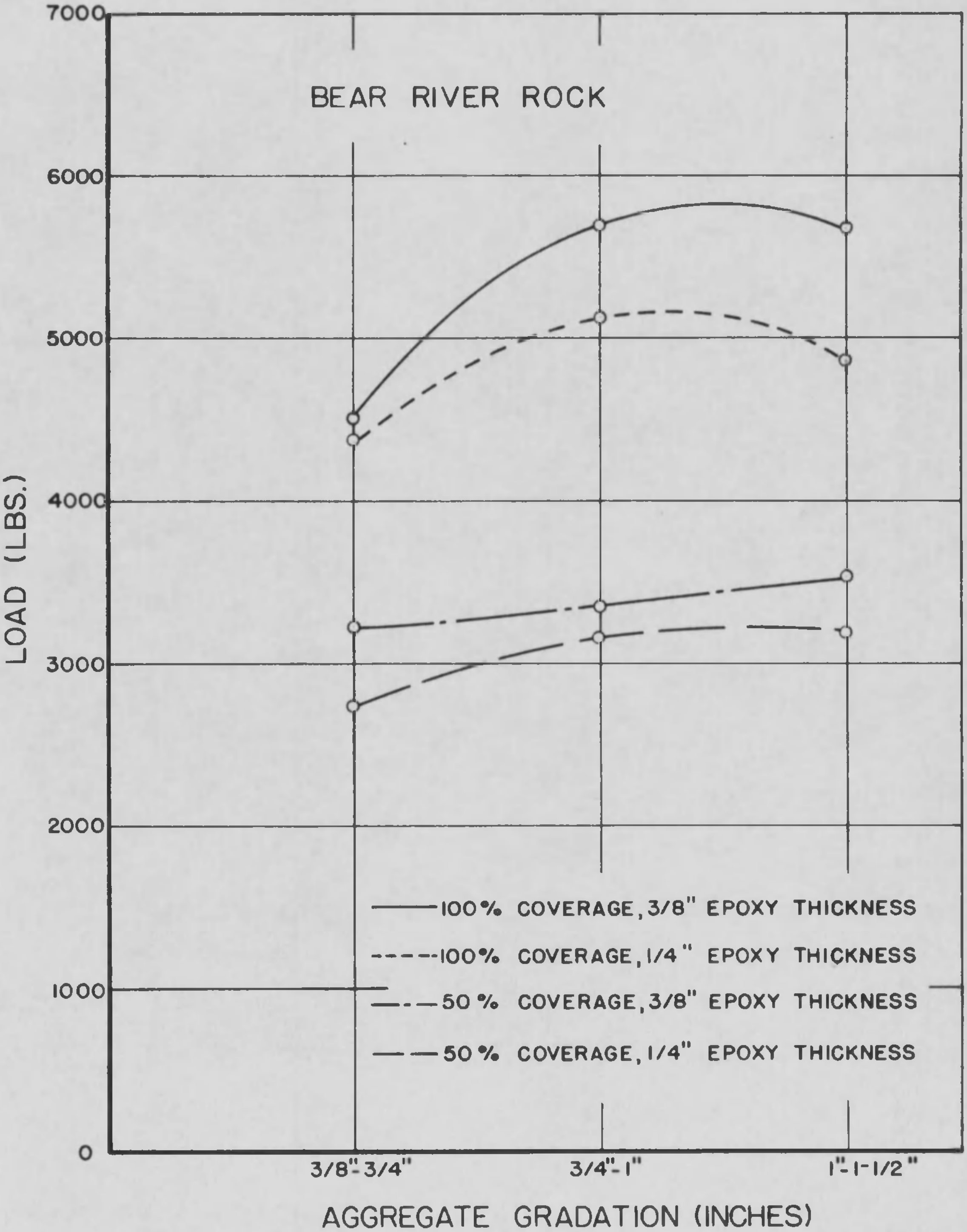


FIGURE 16 LOAD-GRADATION CURVES FOR BEAR RIVER ROCK

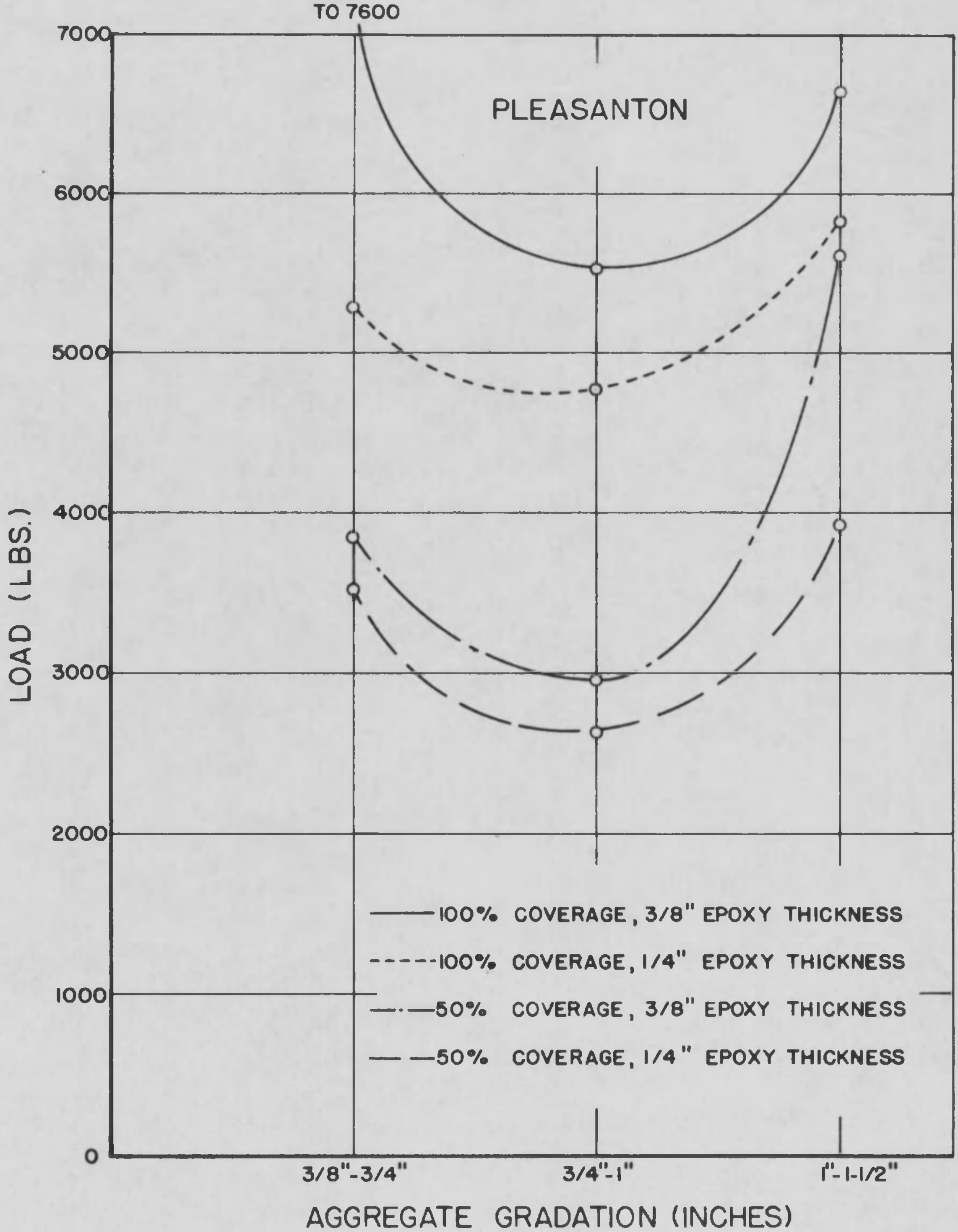


FIGURE 17 LOAD-GRADATION CURVES FOR PLEASANTON

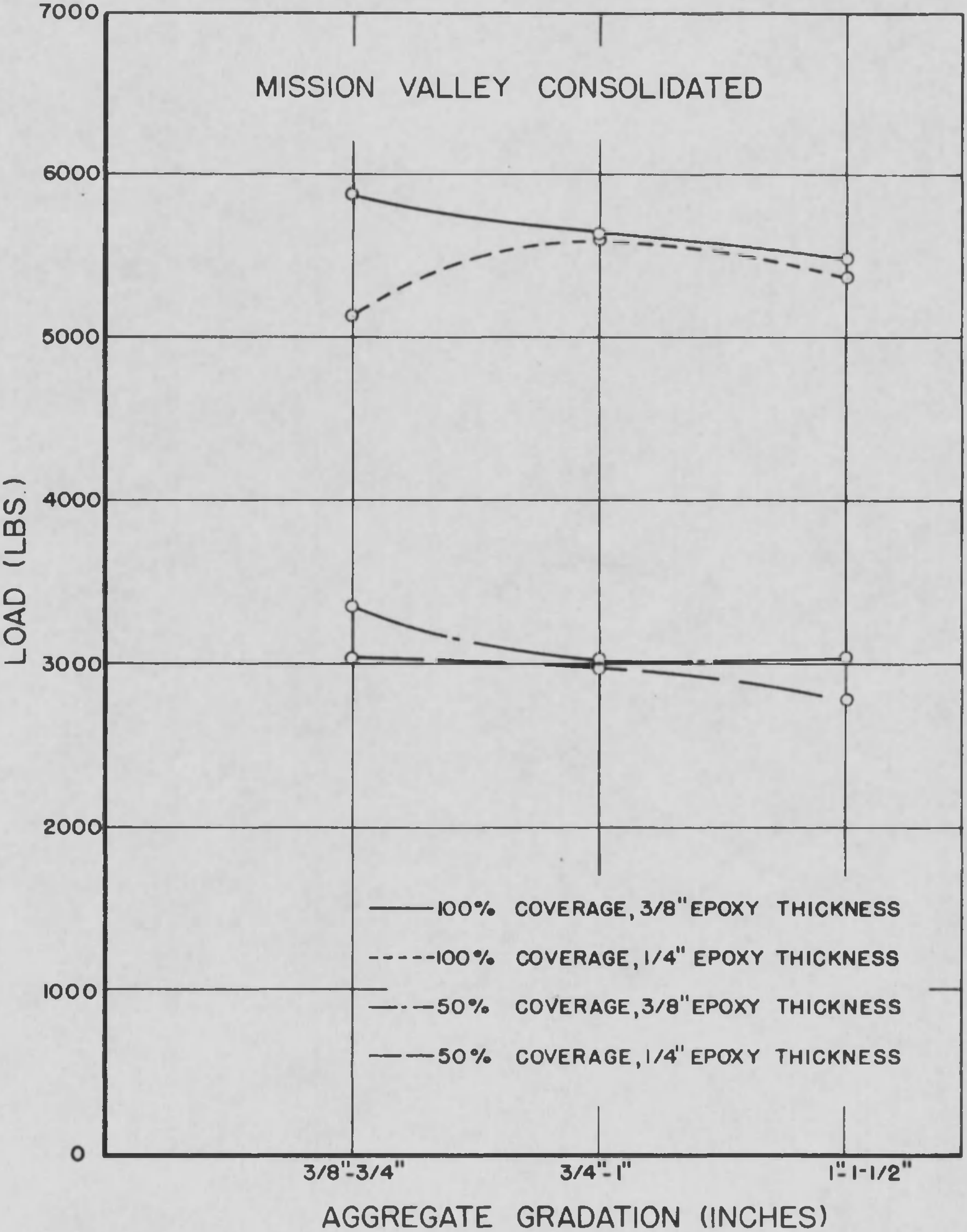


FIGURE 18 LOAD-GRADATION CURVES FOR MISSION VALLEY

CHAPTER 3

TESTS ON COMPOSITE T-BEAMS

3.1. Introduction

Previous tests conducted on composite T-beams at The University of Arizona have demonstrated the effectiveness of an epoxy system as a shear connector. However, the results gave no conclusive indication of the critical shearing stresses which the epoxy system was capable of developing.

The tests described herein were designed specifically to determine the critical shearing stresses developed by the epoxy-bonded aggregate shear connector when subjected to different loading and temperature conditions.

3.2. Description of Test Specimens

The test beam specimens were divided into two groups designated as 12 and 15.

Group 12 consisted of three T-beams which utilized 12I50 rolled steel sections as stems. This group consisted of three composite T-beams designated as 12-A, 12-B, and 12-C.

Group 15 consisted of three T-beams which utilized 15I50 rolled steel sections as stems. This group also consisted of three T-beams. These were designated as 15-A, 15-B, and 15-C.

The dimensions and properties of the T-beams in both groups are shown in Table 3.

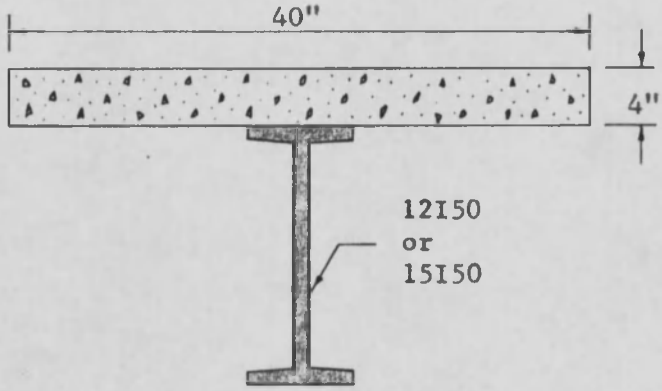
TABLE 3

T-BEAM DIMENSIONS AND PROPERTIES

T-BEAM	LENGTH	STEEL BEAM	SLAB	I IN.^{4}	E_c PSI	f'_c PSI	n	N.A. IN.
*12-A	21'-0"	12I50	40"X4"	985.6	5.85×10^6	10,750	4.96	4.29
12-A	21'-0"	12I50	40"X4"	906.9	4.25×10^6	7,940	6.82	5.08
12-B	21'-0"	12I50	40"X4"	861.2	3.54×10^6	5,814	8.19	5.41
12-C	21'-0"	12I50	40"X4"	854.4	3.44×10^6	6,245	8.43	5.36
*15-A	29'-0"	15I50	40"X4"	1376.5	4.95×10^6	10,525	5.86	5.31
15-A	29'-0"	15I50	40"X4"	1261.2	3.54×10^6	5,814	8.19	6.06
15-B	29'-0"	15I50	40"X4"	1340.9	4.46×10^6	7,795	6.50	5.54
15-C	29'-0"	15I50	40"X4"	1343.3	4.52×10^6	8,030	6.42	5.52

* CALCULATIONS BASED ON CONCRETE PROPERTIES AT TEST TEMPERATURE BELOW 0° F

A typical cross-section of the epoxy-bonded composite T-beam used in these studies is shown below:



The letter designation on each of the beams corresponds to the type of loading and temperature condition to which the T-beam specimens were subjected.

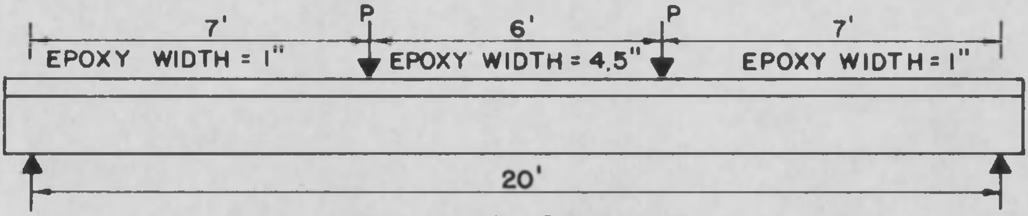
The letter "A" is used to identify the T-beams which were tested as simply supported beams with two point loading and at a temperature below zero degrees Fahrenheit.

The letter "B" identifies the T-beams tested as simply supported beams with two point loading and at a room temperature of $72^{\circ} \pm 5^{\circ}$F.

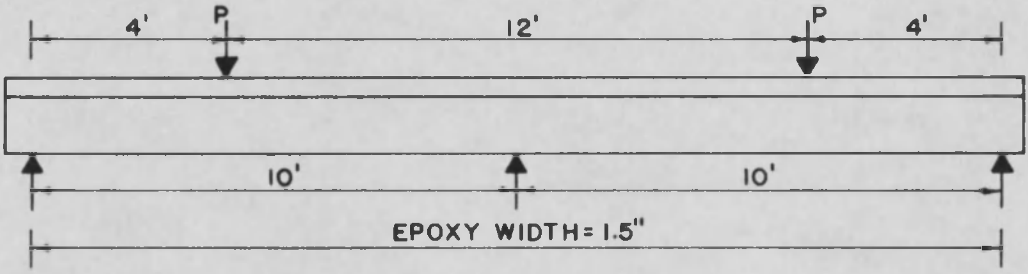
The letter "C" designates that the T-beam was tested as a symmetrical, continuously supported beam with two point loading. T-beams in the "C" series were tested at a room temperature of $72^{\circ} \pm 5^{\circ}$F.

The loading arrangements and support spacing for the various T-beams are shown in Figure 19.

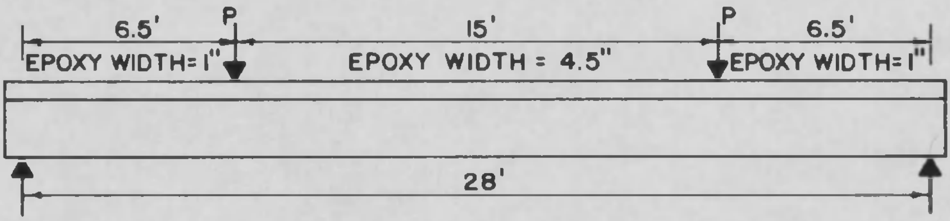
As mentioned previously, these tests were designed specifically to determine the critical shearing stresses which could be developed in



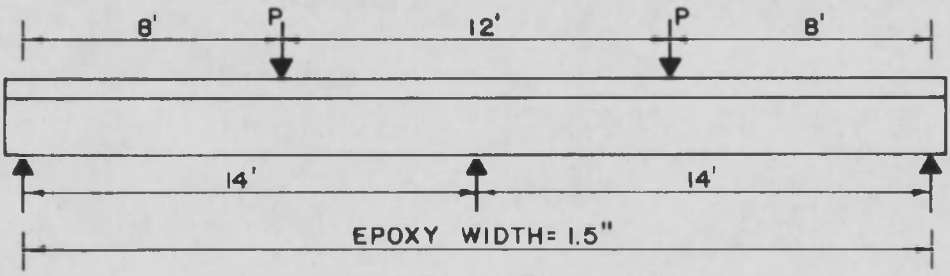
T-BEAMS 12-A, 12-B



T-BEAM 12-C



T-BEAMS 15-A, 15-B



T-BEAM 15-C

FIGURE 19 LOADING ARRANGEMENTS FOR T-BEAMS the epoxy layer. In order to accomplish this, the width of the epoxy layer on the top flange of the rolled steel section was made considerably less than the width of the flange. In normal construction procedures the epoxy bonded aggregate system would likely be applied over the entire area of the top flange. The width was designed in such a manner as to produce a shear failure in the epoxy layer before initial yielding in the rolled steel section or crushing of the concrete slab occurred. Shear stress calculations could then be made while the beam was still in the linear range.

T-beams 12-C and 15-C, which were tested over continuous supports, utilized an epoxy width of 1-½ inches and extended the entire length of the beams. T-beams 12-A, 12-B, 15-A, and 15-B, which were tested as simply supported beams, utilized an epoxy width of one inch in the shear spans and a width of 4.5 inches in the constant moment sections. The reason for the increased width of the epoxy layer in the constant moment sections on the simply supported beams was to insure that a tensile separation between the concrete slab and the rolled steel section did not occur before a shear failure was produced in the shear spans. A bond breaking agent was applied over the remaining portion of the flange to eliminate any bonding produced between the concrete and steel. These shear connectors of reduced width can be seen in Figures 20 and 21.

3.3. Preparation of Contact Surfaces

All steel contact surfaces were sandblasted to remove mill scale and rust. They were then cleaned with methylethyl ketone just prior to the application of the epoxy resin.

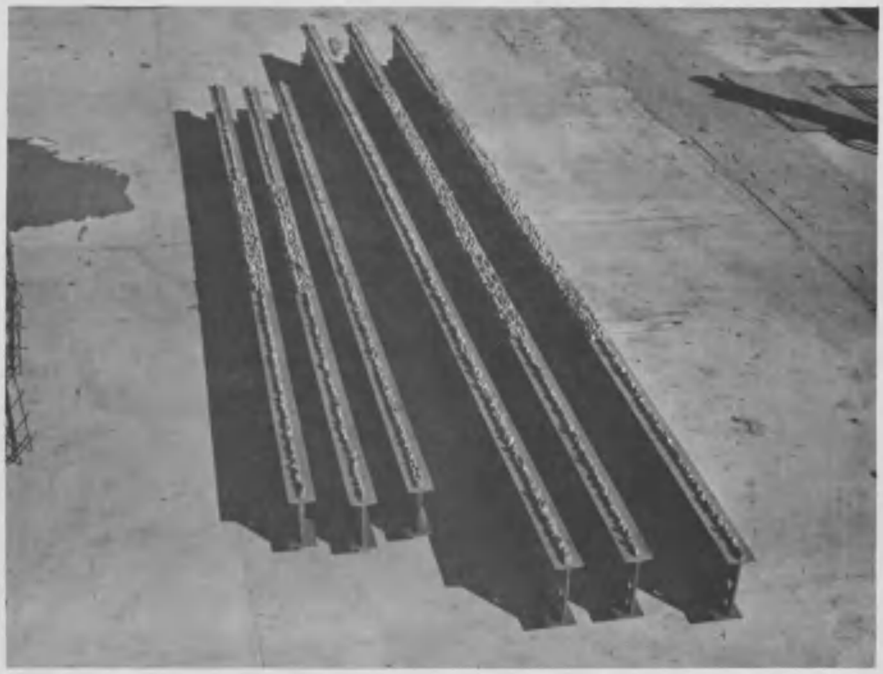


FIGURE 20 BEAMS WITH EPOXY SHEAR CONNECTORS
OF REDUCED WIDTH

FIGURE 21 CLOSE-UP OF SHEAR CONNECTOR

3.4. Preparation of T-Beams

After the steel contact surfaces were cleaned with methylethyl ketone, wooden forms covered with polyethelyne were clamped to the top flanges of the rolled sections to insure proper width of the epoxy layer. These forms were constructed ¼ inch thick to facilitate the application of a uniform epoxy thickness of ¼ inch. The epoxy resin was then applied to the contact surfaces. Washed aggregate was immediately hand placed in the fresh epoxy and pushed down slightly to insure proper bonding of the epoxy to the aggregate. One hundred percent aggregate coverage and an epoxy thickness of ¼ inch was utilized in all of the T-beam test specimens. The aggregate used in these tests consisted of crushed rock with a maximum size of 1½ inches and a minumum size of 1 inch, and was supplied by San Xavier Rock and Sand Company, Tucson, Arizona. The epoxy system was then allowed to cure for a minimum of seven days at room temperature. The beams with the epoxy bonded aggregate system in place were then transported to a local construction company where the concrete slabs were poured. The concrete mix design used in these studies is shown in Appendix A.

All slabs contained shrinkage and temperature reinforcement consisting of 6 x 6 inch welded wire fabric. Tension reinforcement was provided in areas of negative moment in the T-beams which were tested over continuous supports. This tension reinforcement was designed to assure that the shear stress in the epoxy layer reached an assumed critical shear stress value of 2100 psi before the steel reached its yield stress. For convenience, the tension reinforcement was extended the entire length of the continuous beams. Lateral reinforcement was also provided. The slabs were moist cured at the construction site for at least one week.

3.5. Instrumentation

All of the T-beams were instrumented to measure deflection, epoxy bond line slip, and strain distribution at midspan and loading points. A more detailed description of the methods employed to make these measurements is given in section 3.7.

3.6. Testing Procedure

The composite T-beams were tested with a hydraulic ram and loading beam arrangement as shown in Figure 22. This arrangement utilized the structural testing floor in the graduate structures laboratory of The Department of Civil Engineering, The University of Arizona. The hydraulic rams were connected in parallel with each other to insure equal pressures at both loading points. Zero readings were then taken on all gages. The load was then applied in increments and data readings taken after each increment until failure.

The T-beam test setups for the simply supported condition and the continuous condition are shown in Figures 23 and 24 respectively.

T-beams 12-A and 15-A, which were tested at a temperature below zero degrees Fahrenheit, were tested in a manner similar to that shown in Figure 23. Prior to testing an insulated box was constructed around the beam. (See Figures 25 and 26.) A refrigeration unit was used to lower the temperature of the beam specimen to approximately 20°F, dry

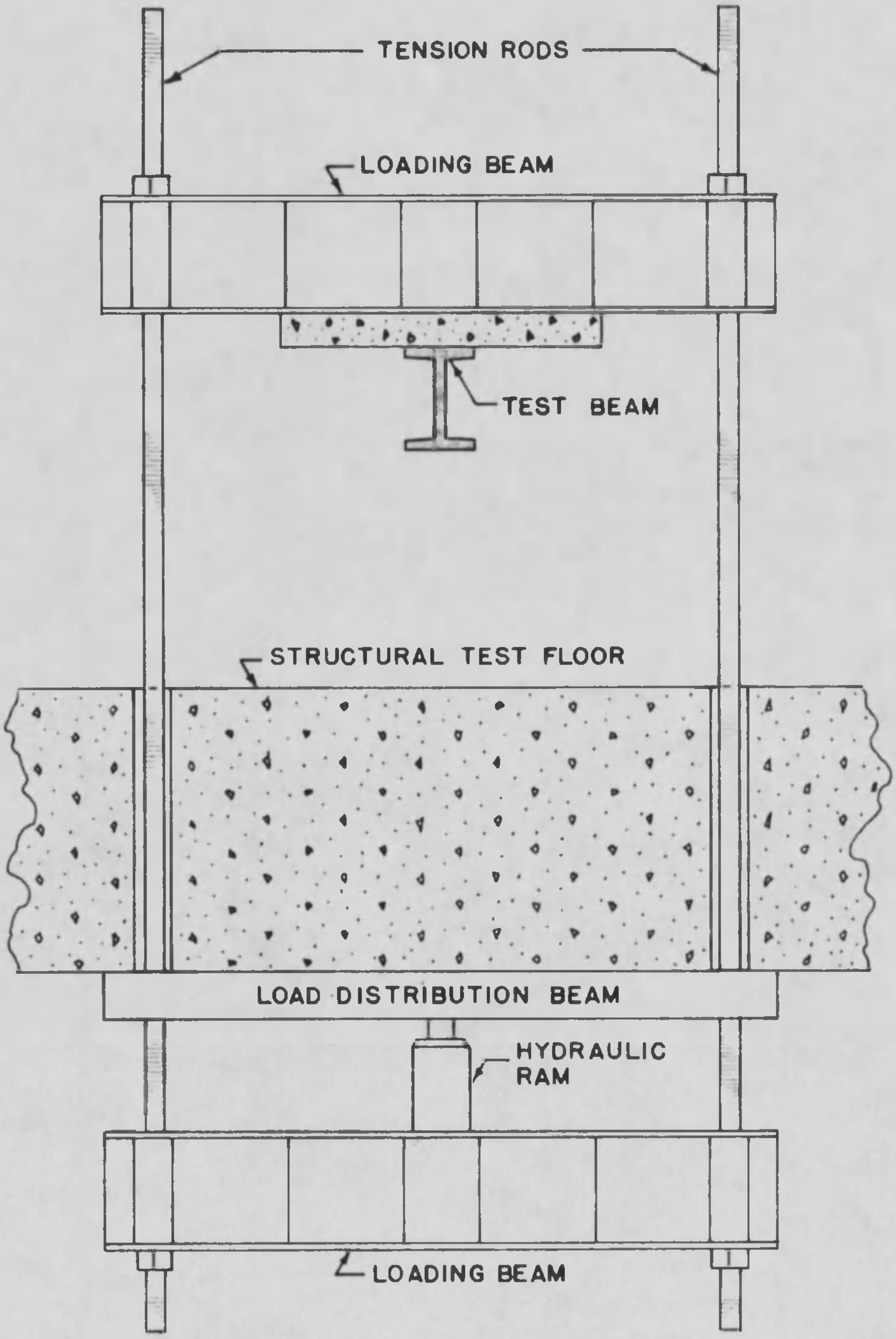


FIGURE 22 TEST SET-UP FOR T-BEAMS

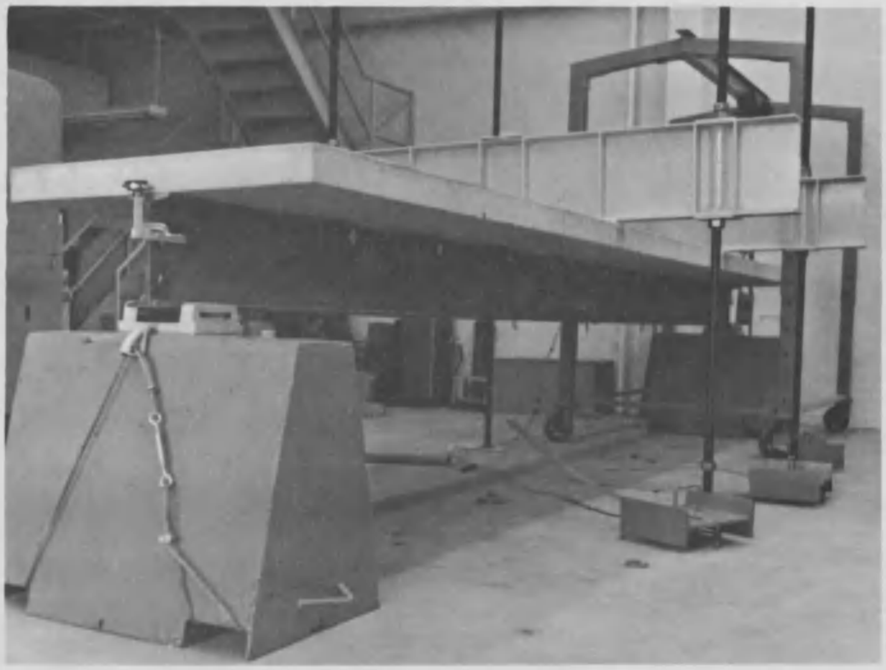


FIGURE 23 TYPICAL TEST SET-UP FOR
SIMPLY SUPPORTED T-BEAM

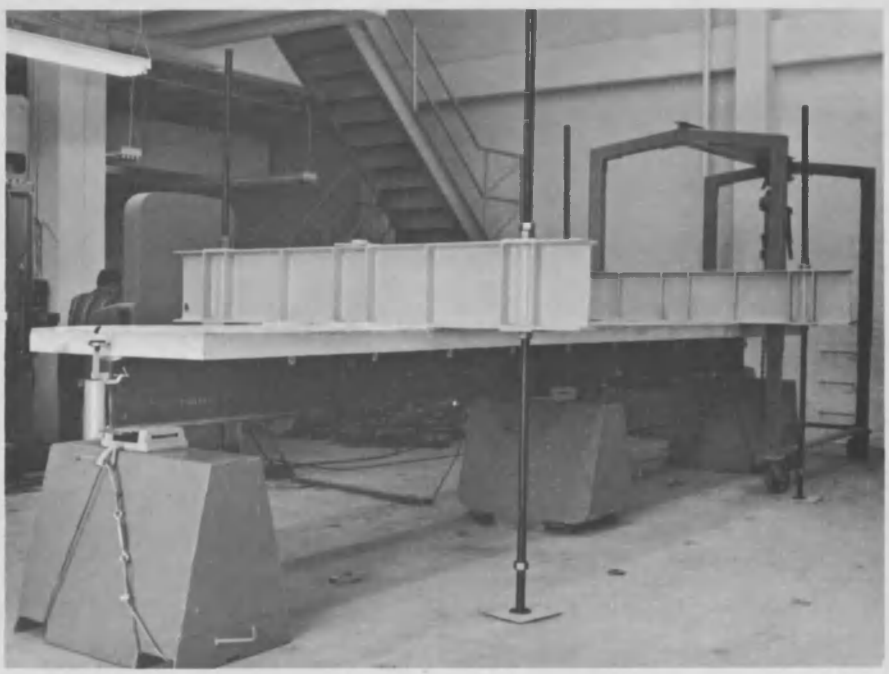


FIGURE 24 TYPICAL TEST SET-UP FOR
CONTINUOUSLY SUPPORTED T-BEAM

FIGURE 25 INSULATED BOX UTILIZED FOR COLD TEMPERATURE TESTING OF T-BEAM 12-A

FIGURE 26 INSULATED BOX UTILIZED FOR COLD TEMPERATURE TESTING OF T-BEAM 15-A

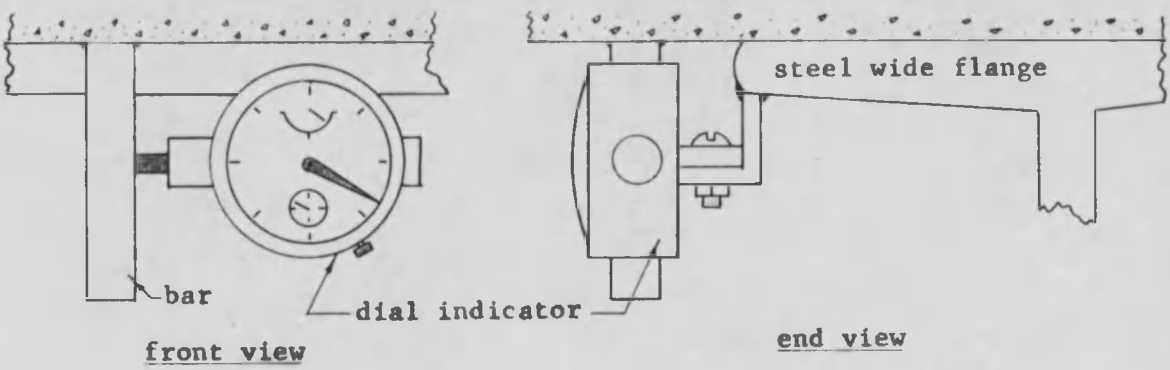
ice was then utilized to drop the temperature below zero degrees Fahrenheit. Thermocouples were attached at various points along the beam to insure that the beam had reached the required temperature. The beam temperature was held below zero degrees Fahrenheit for at least twenty-four hours prior to testing. The insulated box was constructed in such a manner as to allow the beam to be tested without tearing the box away and thus allowing no heat gain during testing. These beams were tested in the same manner as that described above.

3.7. Presentation of Test Data

3.7.1. Load-Slip Curve

Bond line slip as utilized in this thesis is defined as the longitudinal displacement of the concrete slab relative to the rolled steel section stem. Such slip is caused by shear forces which exist in the epoxy layer due to the applied loading.

The bond line slip was measured by ten thousandths dial gages mounted on the steel stem at various points along the length of the beam. The moveable plunger in the dial gage was connected to a steel bar which had been attached to the concrete slab. Any movement between the fixed bar and the fixed dial gage was then taken as the bond line slip. A typical test setup for measuring bond line slip is shown below:



Slip does not occur in a fully composite section; hence, load-slip curves give an indication of the loss of interaction in a T-beam. Since all of the tests were conducted in the linear range, the slip was also an indication of the maximum deformation to which the ¼ inch epoxy layer was subjected before failure.

3.7.2. Test Results for Load-Slip Measurements

Figures 27 through 32 show the load-slip curves for the T-beams of both groups.

The slip at various points along the beams was plotted at different load increments. These curves indicate the approximate maximum slip or deformation to which the epoxy layer was subjected, and show the point at which the shear failure was initiated. Slip readings could not be taken at the ultimate load in all cases because failure had occurred before they could be obtained; however, load-slip curves for the next highest load was plotted and should give an approximate indication of the maximum slip obtained and the point of failure.

The shear in the center spans of the T-beams which utilized simple supports was zero and theoretically produced no slip between the concrete slab and the steel stem. As can be seen from the load-slip curves for these beams the bond line slip in these sections is very small and, for all practical purposes, can be assumed to be zero.

Load-slip curves for T-beams 12-A and 12-B, which were tested as simply supported beams, indicate that maximum values of slip obtained in the epoxy layer before failure to be 27.5 ten thousandths (0.00275) inch and 25 ten thousandths (0.0025) inch respectively. T-beam 12-B failed at a load of 19.68 kips before slip readings could be taken, therefore, the maximum slip was not recorded. However, a projection of the load-slip curves for the other loads indicate that a maximum slip value of 35 ten thousandths (0.0035) inch could have possibly been reached before failure.

Load-slip curves for T-beams 15-A and 15-B, which were tested as simply supported beams, indicate the maximum values of slip before failure to be 38 ten thousandths (0.0038) inch and 62.1 ten thousandtsh (0.00621) inch for T-beams 15-A and 15-B respectively. T-beam 15-A failed at a load of 19.88 kips before slip readings could be taken. However, projection of the load-slip curves for the other load increments indicate that an approximate value of maximum slip could be assumed to be 45 ten thousandths (0.0045) inch.

T-beams 12-A and 15-A were tested at a temperature below zero degrees Fahrenheit while T-beams 12-B and 15-B were tested at a room temperature of $72^{o} \pm 5^{o}F$. As a result, comparisons could be made to determine the effects of temperature on the epoxy bond line slip. These comparisons were conducted for the beams of both groups and were made by comparing the maximum slips produced in the epoxy layer at various load increments. The percent of difference was then calculated at each load increment. These results are shown in Table 4. As can be seen from the results the percentages of difference ranged from an increase in slip of 8.3 percent to a decrease in slip of 26.5 percent. The general tendency of these results might indicate that the difference in temperatures produced a decrease in epoxy bond line slip; however, past results from tests conducted to determine epoxy strength properties indicate that differences of 25 percent exist in the properties for different batches of the same epoxy formulation. Since the differences of slip for the beams tested at room temperature and the beams tested at a temperature below zero degrees Fahrenheit fell within this approximate 25 percent range, it appears that the temperature had little or no effect on the deformations in the epoxy layer.

T-beam 12-C, which was tested over continuous supports, produced a maximum slip of 87.6 ten thousandths (0.00876) inch before a shear failure was produced in the epoxy layer over the center support. Comparing the maximum slips of T-beams 12-A and 12-B with those produced in T-beam 12-C indicates that an increase of approximately 160 percent in the maximum slip was produced when the beam was tested over continuous supports. This increase was due in part to differences in the physical properties of the epoxy. It is also thought that the difference in the radii of curvature has a large effect on the ability of the epoxy layer to deform. A negative radius of curvature is produced over the center support of the continuous beams and, hence, there is no tendency for the concrete slab and the steel stem to separate vertically. Therefore, it is thought that the maximum slip produced in the simply supported beams is also dependent on the vertical forces existing in the epoxy layer between the concrete slab and the steel stem. No attempts were made to determine the effect of these vertical forces on the maximum slip; however, they can be seen to have a definite effect.

Load-slip curves for T-beams 15-C, which was tested over continuous supports, indicate the maximum value of slip to be 59.2 ten thousandths (0.00592) inch. Comparing the load-slip curves of T-beams

15-B and 15-C indicates a general increase in slips for the beam tested over continuous supports but no increase in the maximum slips produced in the epoxy layer. These results are not in accordance with those obtained from T-beam 12-C, however, it is thought that the increase in span lengths for T-beam 15-C affected the radii of curvature in such a manner as to produce a different type of failure from that produced in T-beam 12-C. It should be noted that failure was produced mid-way between the center support and the load rather than over the center support as can be seen in T-beam 12-C. Whether or not the vertical forces existing in the epoxy layer at this point had any effect on the maximum slip could not be determined; however, this could possibly be the reason for the lower slip values obtained from T-beam 15-C.

3.7.3. Load-Deflection Curves

Load-deflection curves for T-beams 12-A, 12-B, 15-A, and 15-B are shown in Figures 33 through 36, respectively.

The load-deflection curves indicate the ultimate strength and toughness developed in the T-beams at the time of failure. They also give some indication to the degree of interaction obtained. All of the T-beams tested in these studies were designed to fail in shear before initial yielding in the steel stem or crushing of the concrete slab occurred. For this reason, the load-deflection curves, as utilized here, will not give a good indication of the ultimate strength and toughness which could have been developed by the T-beams had an epoxy width equal to the width of the flange been used.

The experimental deflections were obtained by utilizing a cantilevered steel strip device which was suspended from the bottom flange of the steel stem. Readings were taken with a SR-4 strain gage which had been attached to the underside of the cantilevered steel strip. The readings were then converted to deflections by means of calibration charts which had been prepared for each gage. A typical test setup for the cantilevered deflection gages can be seen in Figure 23.

3.7.4. Results of Load-Deflection Measurements

As can be seen from the experimental load-deflection curves there is a scattering of points. However, this scattering is in the order of magnitude between 0.01 and 0.03 inches and is thought to be quite negligible. This scattering of points was due in part to the slight tipping action of the T-beams as they were being tested. This tipping was probably due to small differences of load in the tension rods of the loading mechanism, thus resulting in a slight tipping of the loading beams.

The experimental deflection points were plotted as obtained from the test results,and the best straight line drawn through them. This experimental curve was then corrected by shifting it in such a manner so as to pass through the zero point. The theoretical curve was calculated assuming that 100 percent interaction existed between the concrete slab and steel stem.

Comparison of the slopes of the theoretical deflection curves with the experimental deflection curves show a decrease in slope of 20.4, 8.93, and 16.5 percent for T-beams 12-A, 15-A, and 15-B, respectively. This percentage of decrease in slope does not indicate the percentage of interaction lost in the test beams due to the fact that they were not compared with theoretical curves of zero percent interaction. These percentages, however, do indicate a definite loss of interaction. These results are very acceptable considering that an epoxy width of only one inch was utilized in the shear spans of these beams.

Comparison of the theoretical and experimental slopes for the deflection curves of T-beam 12-B shows an increase in slope of 8.29 percent. These results would indicate an increase in interaction; however, an insufficient number of points were obtained to determine a good curve and it is thought that these results should be disregarded.

3.7.5. Ultimate Shearing Stresses Developed in Epoxy Layer

The ultimate shearing stresses developed in the epoxy layer were determined specifically to aid in the development of design criteria and specifications for epoxy bonded aggregates when used as a shear connector in composite construction.

Failure of the T-beams in the linear range facilitated the problem of calculating the developed shear stresses by making possible the use of the elementary strength of materials formula for determining shear stresses.

T-beams 12-A, 12-B, 15-A, and 15-B, which were tested as simply supported beams, developed maximum shear stresses of 1438, 1520, 1303, and 1722 psi, respectively, before failure in the epoxy layer was produced. T-beams 12-A and 15-A were tested at a temperature below zero degrees Fahrenheit while T-beams 12-B and 15-B were tested at a room temperature of $72^{\circ} \pm 5^{\circ}$F. Comparison of the ultimate shearing stresses developed in the epoxy layer for T-beams 12-A and 12-B indicates that the decrease in temperature produced a loss in shear strength of 92 psi or 6.05 percent. The same comparison when made between T-beams 15-A and 15-B indicate that a loss of 419 psi or 24.3 percent was caused by the decrease in temperature.

Shear stress calculations performed on T-beams 12-C and 15-C, which were tested over continuous supports, show that shear stresses of 2240 and 2220 psi were developed in the epoxy layer before failure. Comparing the ultimate shearing stresses of T-beams 12-C and 15-C with those developed in T-beams 12-B and 15-B indicate that an increase in shear stress of 47.4 and 29.5 percent was obtained when the beams were tested over continuous supports.

Shear stress calculations on T-beams 12-A and 15-A were also made utilizing concrete properties as determined from tests on control cylinders conducted at room temperature. These results indicate that differences of 2.1 and 13.7 percent were obtained when compared to the results obtained by utilizing cold temperature properties.

The test results for the T-beams of both groups are shown in Table 5.

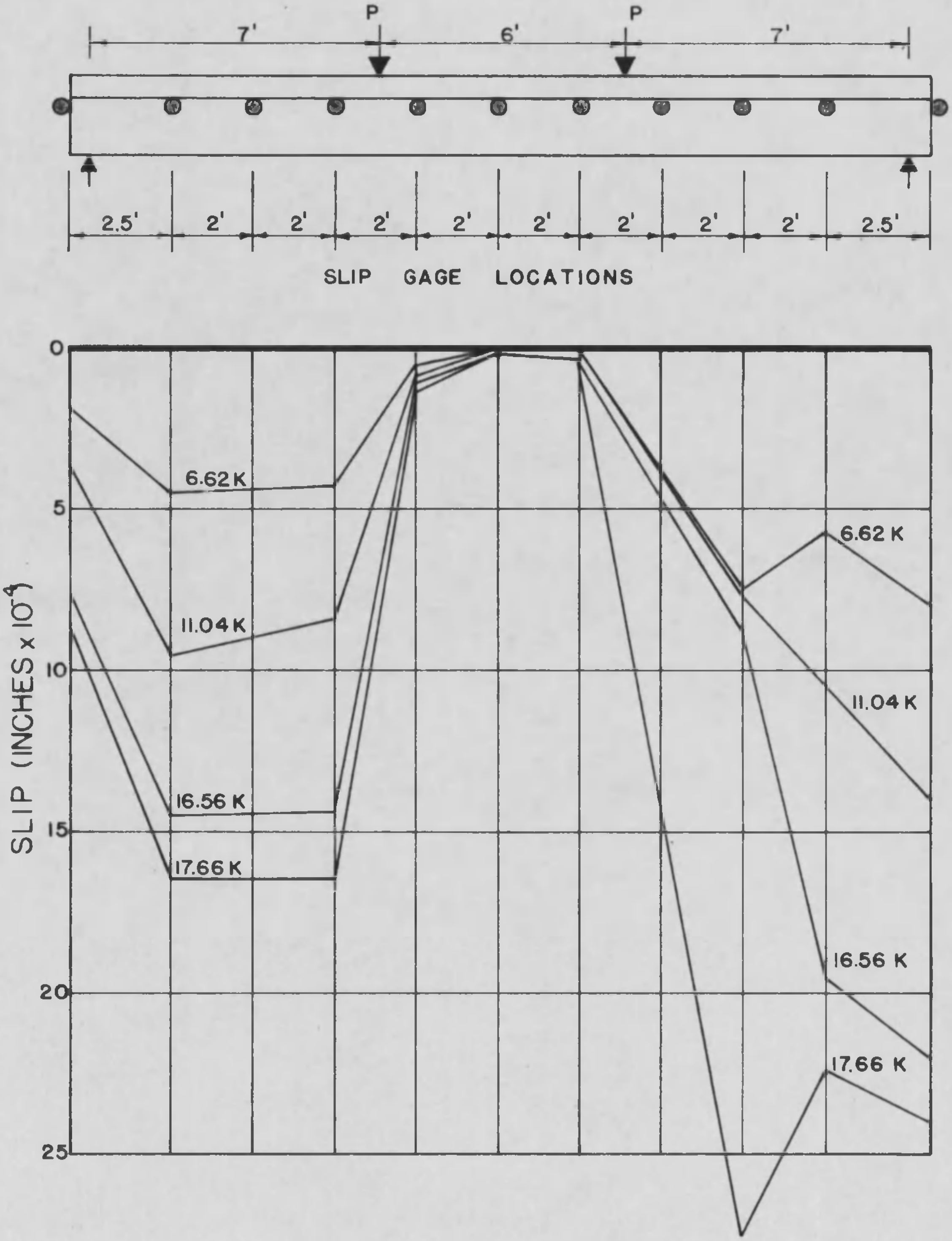


FIGURE 27 LOAD-SLIP CURVES FOR T-BEAM 12-A

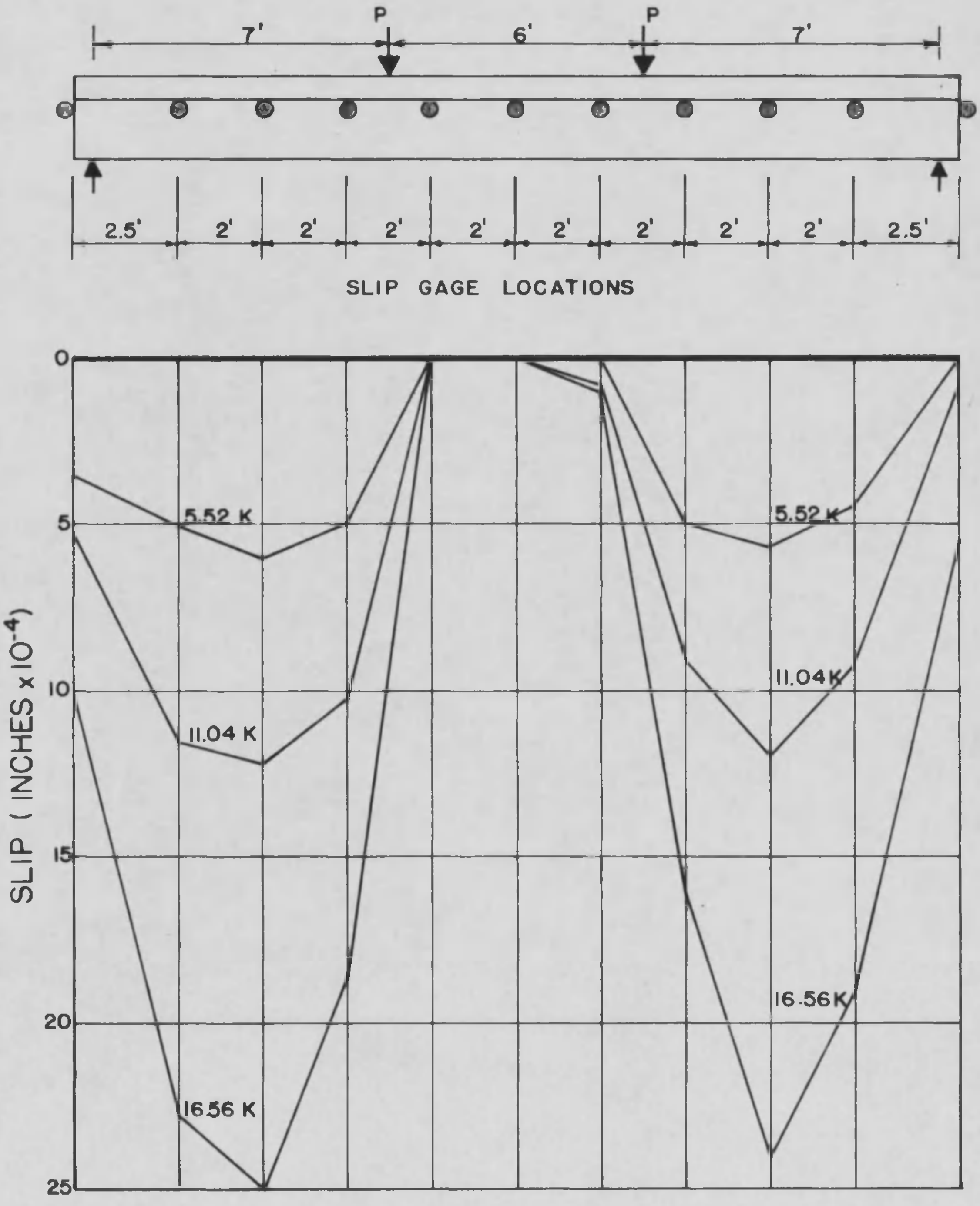


FIGURE 28 LOAD-SLIP CURVES FOR T-BEAM 12-B

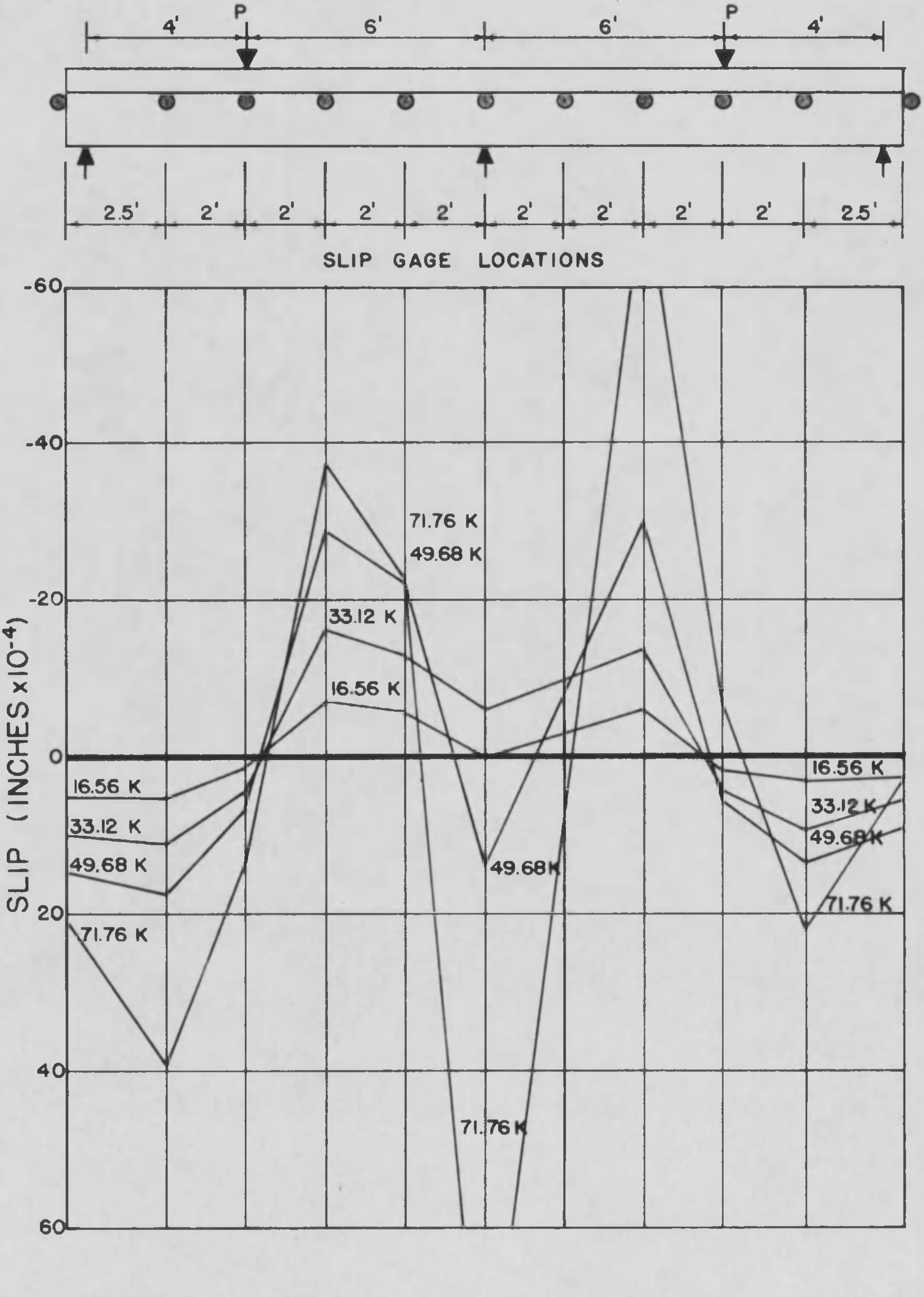


FIGURE 29 LOAD-SLIP CURVES FOR T-BEAM 12-C

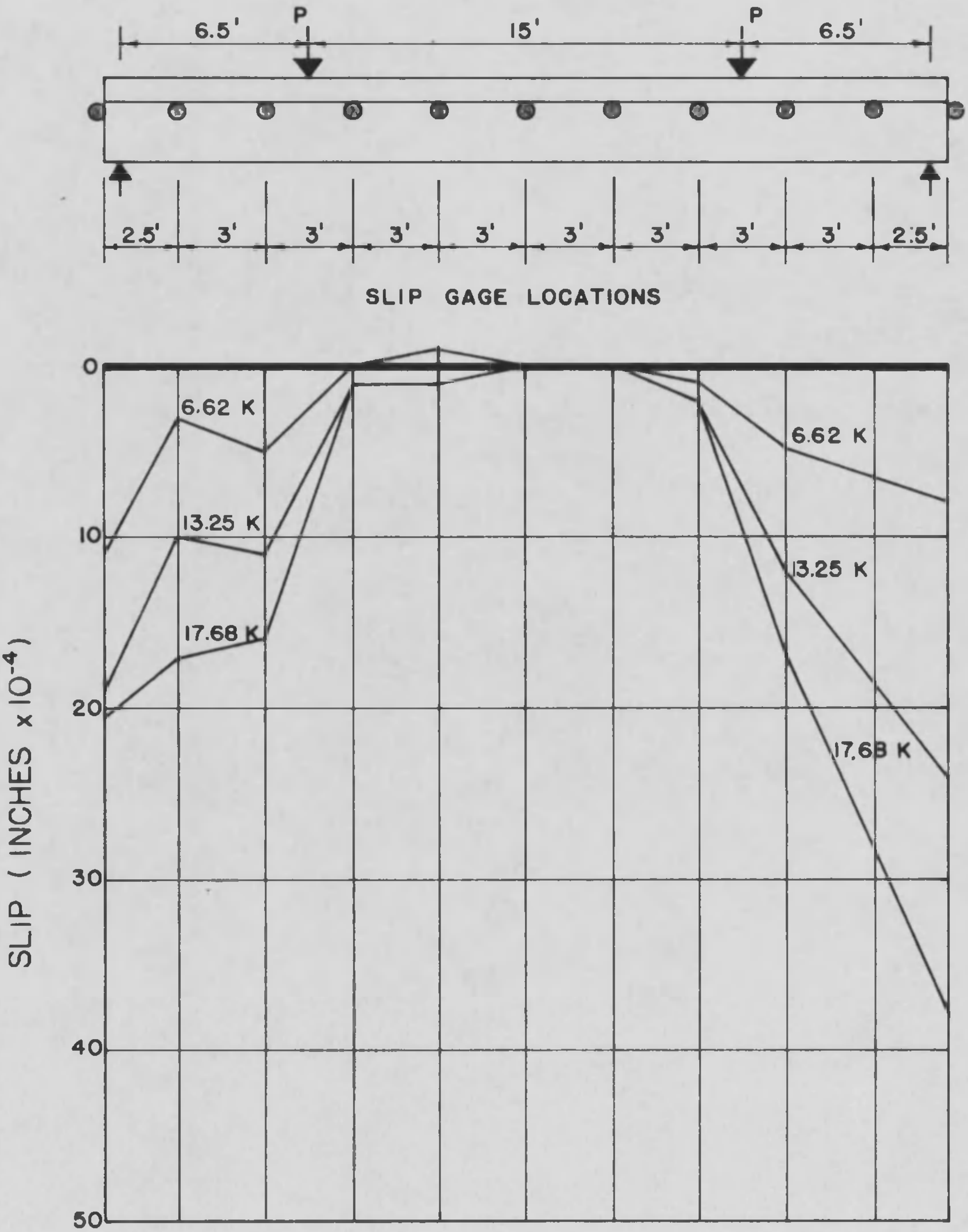


FIGURE 30 LOAD-SLIP CURVES FOR T-BEAM 15-A

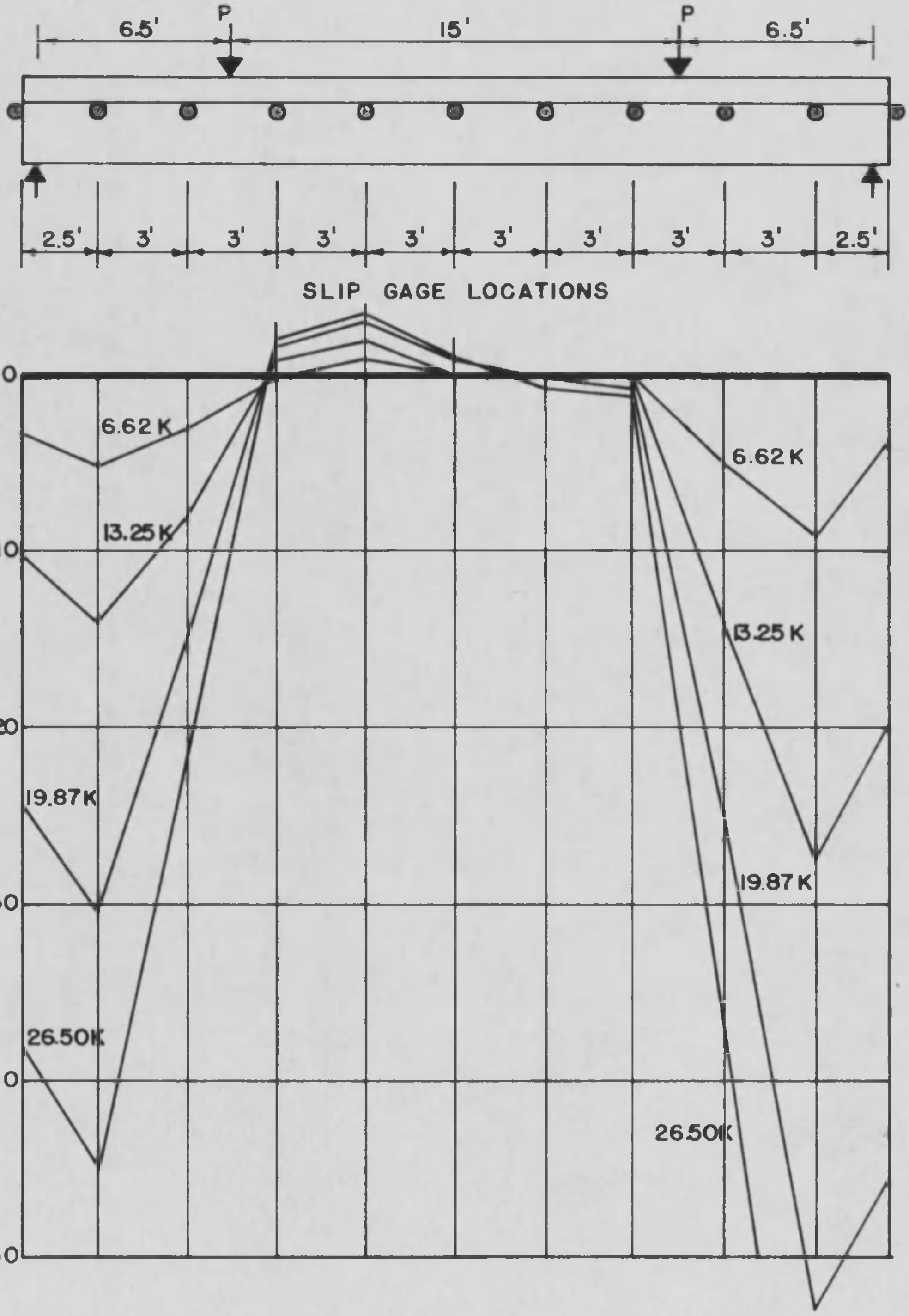


FIGURE 31 LOAD-SLIP CURVES FOR T-BEAM 15-B

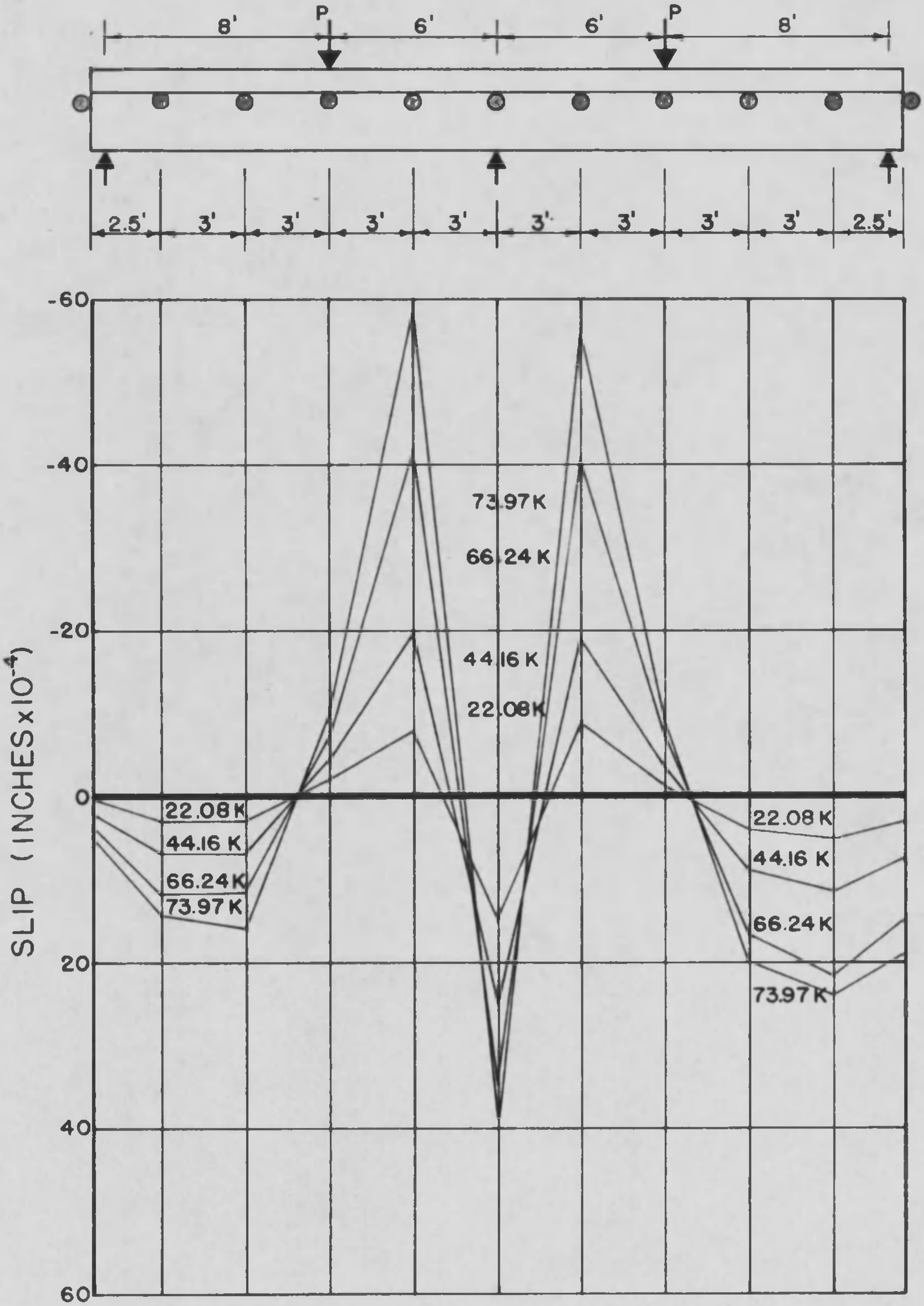


FIGURE 32 LOAD-SLIP CURVES FOR T-BEAM 15-C

TABLE 4

COMPARISON OF MAXIMUM SLIPS AT VARIOUS LOADS

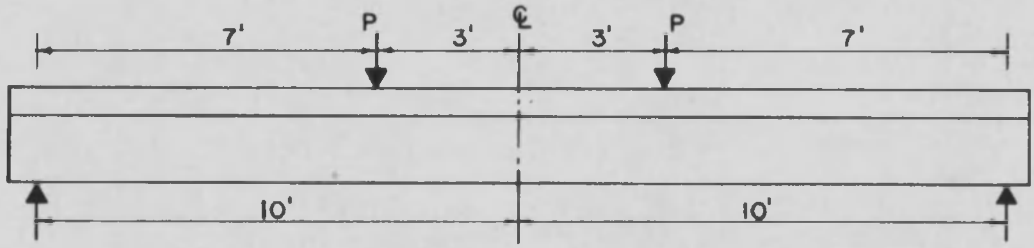
LOAD (P) KIPS	MAXIMUM SLIP T–BEAM 12-B INCHES X 10^{-4}	MAXIMUM SLIP T–BEAM 12-A INCHES X 10^{-4}	% DIFFERENCE $\frac{(12\text{-}B)-(12\text{-}A)}{(12\text{-}B)}$
5.52	6.0	6.5	-8.33
11.04	12.2	14.0	-14.75
16.56	25.0	22.0	12.0
ULTIMATE	35.0	27.5	21.4

LOAD (P) KIPS	MAXIMUM SLIP T–BEAM 15-B INCHES X 10^{-4}	MAXIMUM SLIP T–BEAM 15-A INCHES X 10^{-4}	% DIFFERENCE $\frac{(15\text{-}B)-(15\text{-}A)}{(15\text{-}B)}$
4.42	6.0	5.0	16.7
8.84	14.0	11.0	21.4
13.25	27.5	24.0	12.7
17.76	44.0	38.0	13.6
ULTIMATE	61.2	45.0	26.5

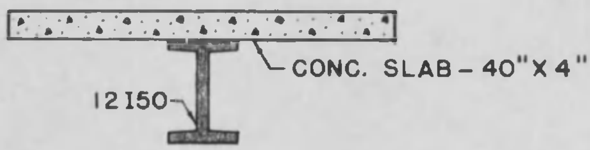
<u>TEMPERATURE OF BEAMS DURING TEST</u>

T–BEAMS 12-A, 15-A : BELOW 0°F

T–BEAMS 12-B, 15-B : 72° ± 5° F



LOADING ARRANGEMENT



CROSS-SECTION

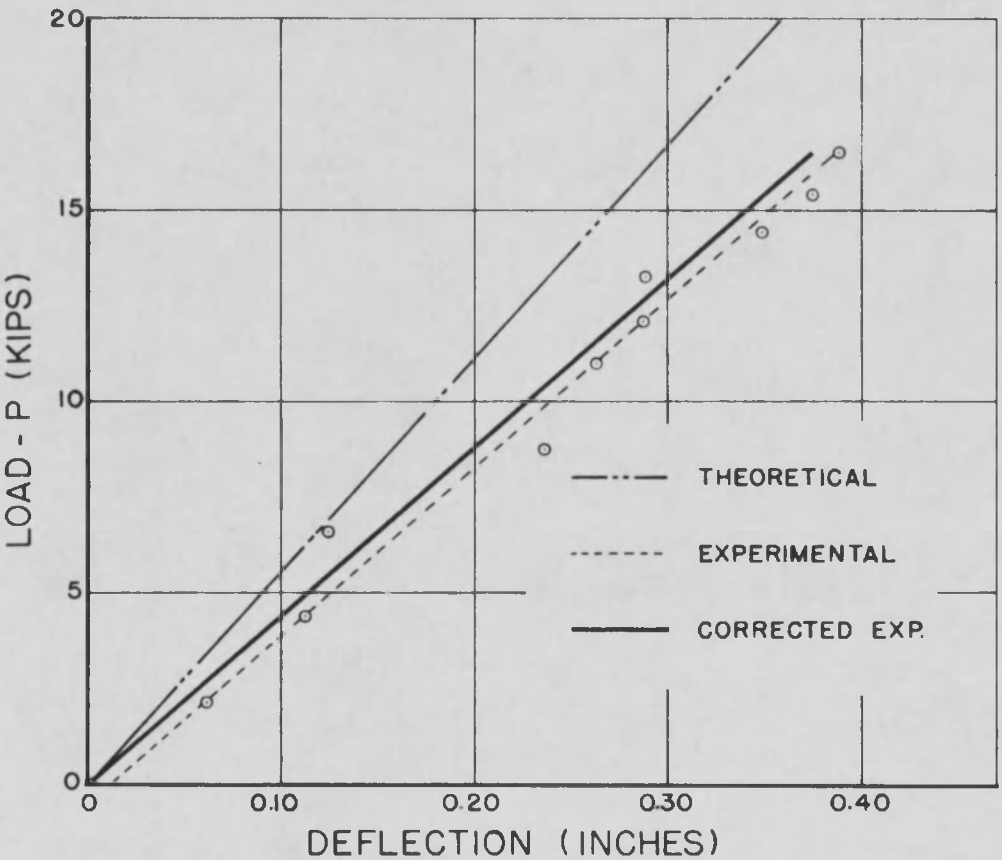


FIGURE 33 LOAD-DEFLECTION CURVE FOR T-BEAM 12-A

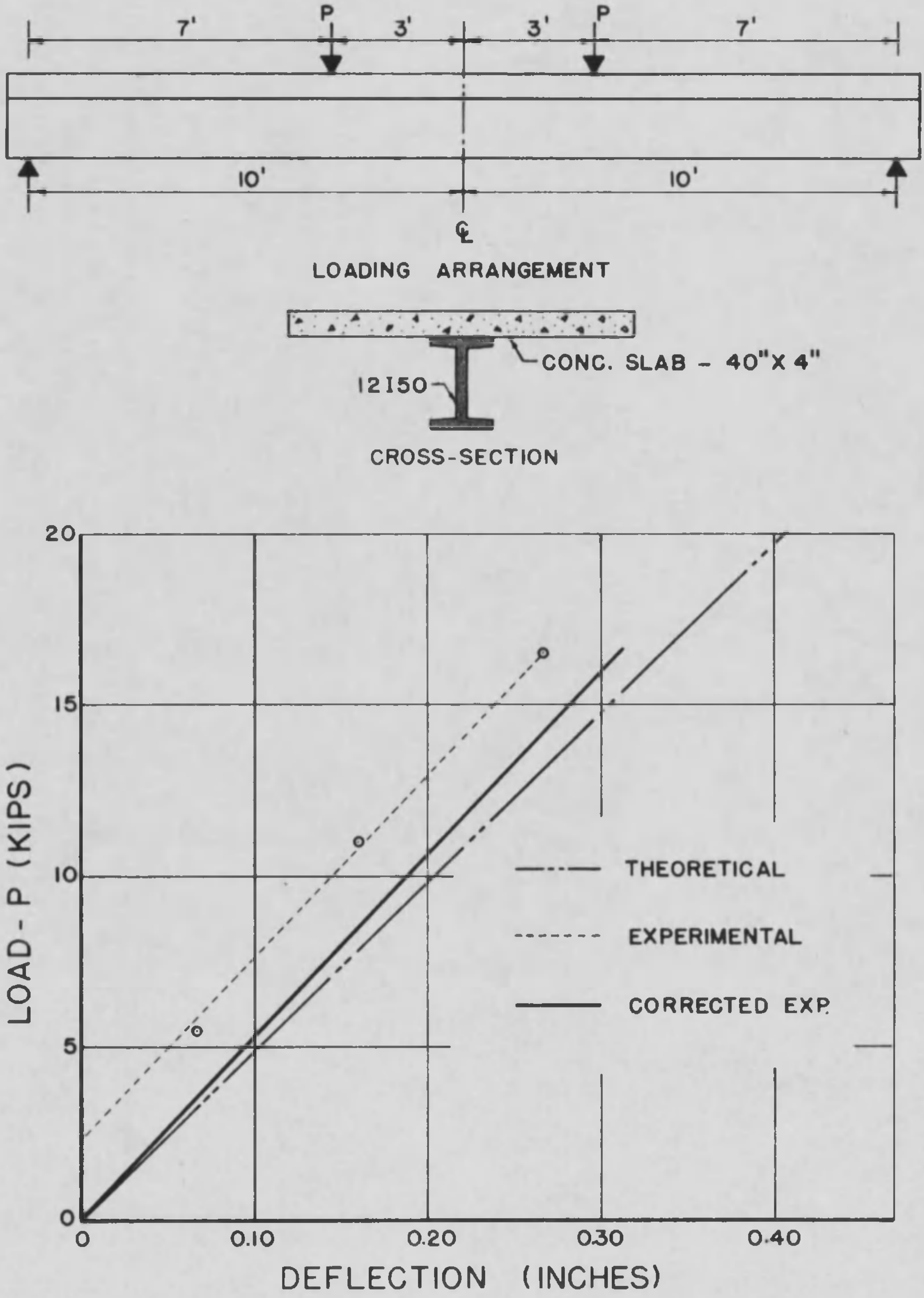


FIGURE 34 LOAD-DEFLECTION CURVE FOR T-BEAM 12-B

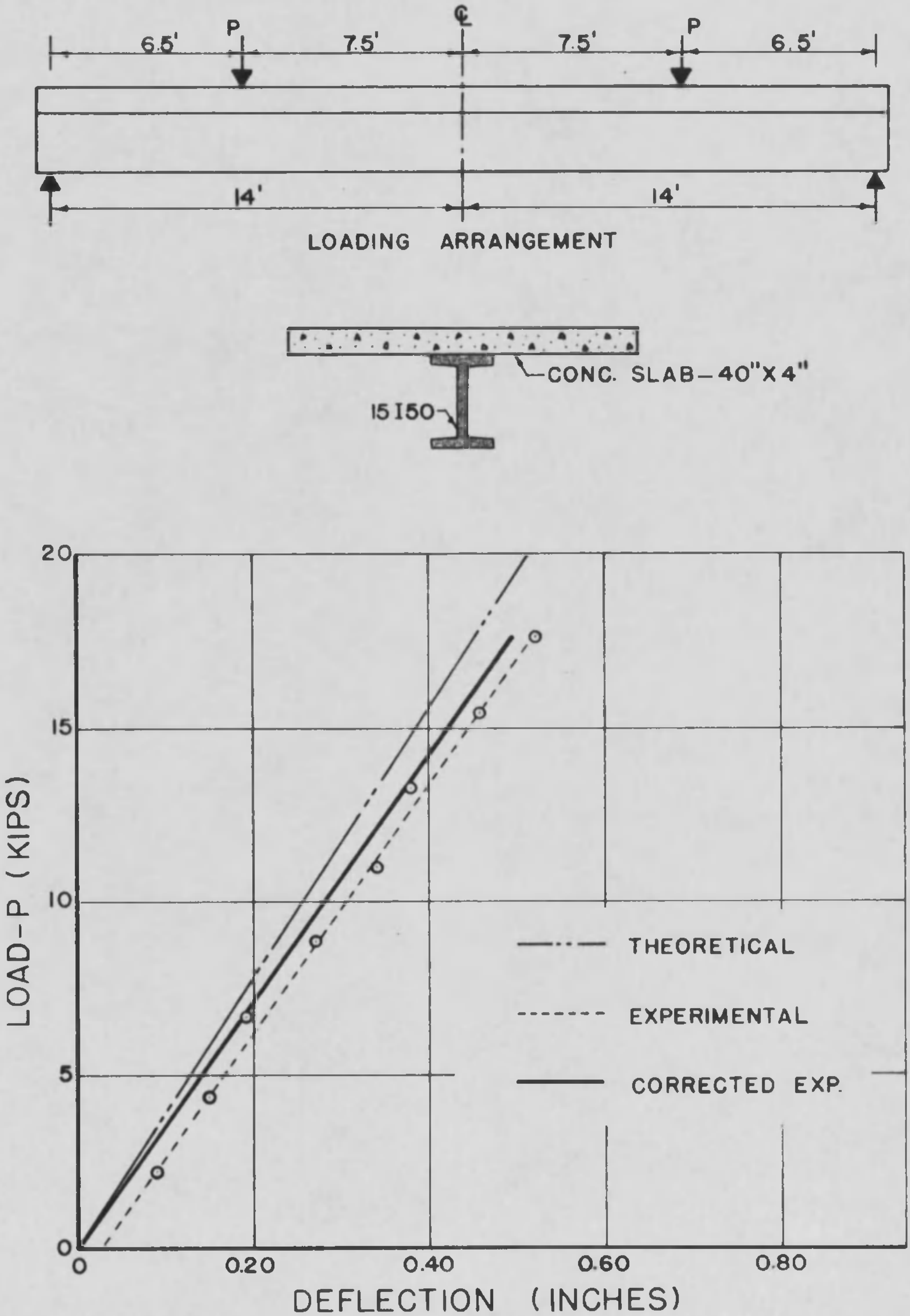
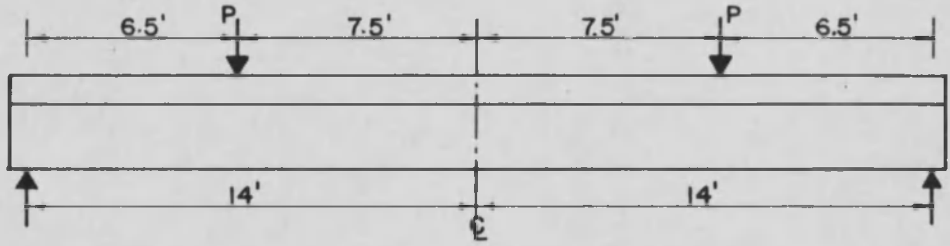


FIGURE 35 LOAD-DEFLECTION CURVE FOR T-BEAM 15-A



LOADING ARRANGEMENT

CROSS-SECTION

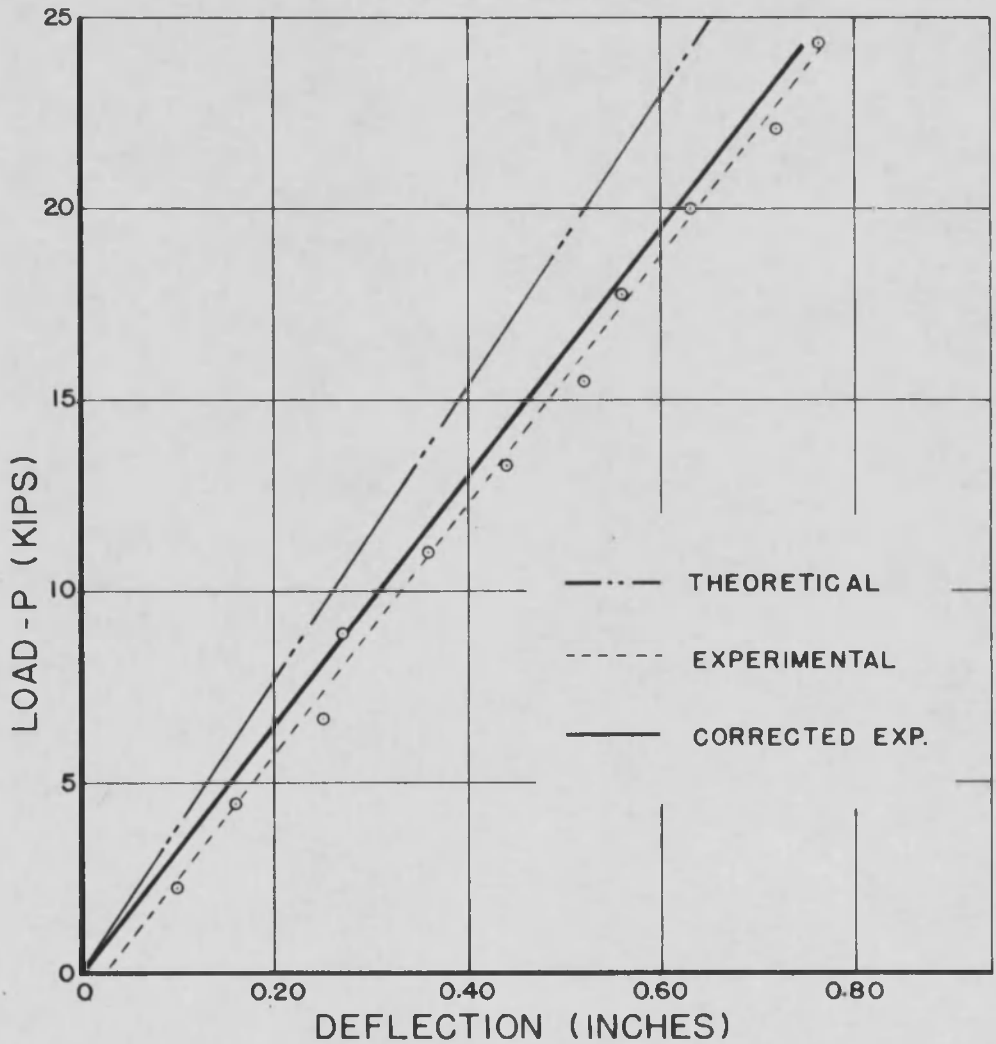


FIGURE 36 LOAD-DEFLECTION CURVE FOR T-BEAM 15-B

TABLE 5

RESULTS OF T-BEAM TESTS

T-BEAM	ULT. LOAD KIPS	ULT. SHEAR FORCE KIPS	I IN.^{4}	M IN.^{3}	b IN.	ULT. SHEAR STRESS PSI	TYPE OF FAILURE
*12-A	17.66	17.66	985.6	80.2	1.00	1438	SHEAR
12-A	17.66	17.66	906.9	72.3	1.00	1408	
12-B	19.68	19.68	861.2	66.6	1.00	1520	SHEAR
12-C	71.76	39.30	854.4	74.7	1.50	2240	SHEAR
*15-A	19.88	19.88	1376.5	90.4	1.00	1303	SHEAR
15-A	19.88	19.88	1261.2	94.1	1.00	1482	
15-B	26.50	26.50	1340.9	87.1	1.00	1722	SHEAR
15-C	72.04	51.08	1343.3	87.7	1.50	2220	SHEAR

* RESULTS BASED ON CONCRETE PROPERTIES AT TEST TEMPERATURE BELOW 0°F

CHAPTER 4

MATERIAL SPECIFICATIONS

4.1. Introduction

Before design criteria can be developed concerning the use of epoxy bonded aggregates as a shear connector, it will be necessary to establish certain material properties and controls. These properties will be established for the epoxy formulation and aggregates which will be utilized in this system.

4.2. Aggregates

All aggregates when used in an epoxy bonded aggregate system shall conform to the following specifications concerning material and physical properties.

(A) The percentage of wear shall not exceed a value of 30 when tested in accordance with the A.A.S.H.O. Standard Method of test for Abrasion of Coarse Aggregate by use of the Los Angeles Machine, T 96 (A.S.T.M. C 131).

(B) The weighed loss of the aggregate shall not exceed 3.0 percent when subjected to five alternations of the sodium sulfate soundness test. Testing of the aggregate shall conform with the A.A.S.H.O. Standard Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate, T85 (A.S.T.M. C127).

(C) Preferably, the aggregate should have absorption characteristics of less than 1.5 percent when tested in accordance with the A.A.S.H.O. Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate, T85 (A.S.T.M. C127).

(D) The aggregate size shall in all cases be greater than one inch, but shall not exceed a maximum size of 1½ inches.

(E) The aggregate in all cases shall be of a highly crushed variety. The particles shall be angular in shape and shall have a rough surface such as will thoroughly bond with the epoxy and concrete.

4.3. Epoxy Resin

(A) The epoxy formulation shall consist of the ingredients and their respecting proportions as shown for the three component system on page 16 of this thesis.

(B) The epoxy resin formulation shall develop a tensile adhesive strength of not less than 4500 psi, when tested by the method as described in section 2.3.1. of this thesis. No less than five specimens shall be tested in determination of an average strength value.

(C) The epoxy resin formulation shall develop a lap-shear adhesive strength of not less than 1800 psi, when tested by the method as described in section 2.3.1. of this thesis. No less than three specimens shall be tested in determining an average strength value.

(D) The thickness of the epoxy layer in all cases shall not be less than 1/4 inch nor greater than 1/2 inch. Preferably, the epoxy thickness shall be 3/8 inch.

CHAPTER 5

DESIGN SPECIFICATIONS

5.1. Introduction

Design specifications for epoxy bonded aggregates when used as a shear connector in composite construction have been prepared by modifying the present A.A.S.H.O. specifications concerning highway bridges. These modified specifications are shown below and utilize the same numbering system as that used in the present A.A.S.H.O. specifications (the American Association of State Highway Officials, Standard Specifications for Highway Bridges, Eighth Edition, 1961, Division 1, Section 9). A commentary on the modifications and additions made in the specifications is given in section 5.3 of this thesis.

5.2. Modified Specifications

Section 9 - Composite Beams

1.9.1. General

This section pertains to structures composed of steel beams with concrete slabs connected by shear connectors.

General specifications pertaining to the design of concrete and steel structures shall apply to structures utilizing composite beams where such specifications are applicable. Composite beams and slabs shall be designed and the stresses computed by the composite moment of inertia method and shall be consistent with the predetermined properties of the various materials used.

The modulus of elasticity of all grades of structural steel shall be taken to be 29,000,000 pounds per square inch.

The ratio of the moduli of elasticity of steel to those of concrete of various design strengths shall be in accordance with the provisions of Article 1.4.11. (A.A.S.H.O., Standard Specifications for Highway Bridges, 1961).

The effect of creep shall be considered in the design of composite beams which have dead loads acting on the composite section. In such structures, stresses and horizontal shears produced by dead loads acting on the composite section shall be computed for "n" as given in Article 1.4.11 (A.A.S.H.O., Standard Specifications for Highway Bridges, 1961) or for this value multiplied by 3, whichever gives the higher stresses and shears.

If concrete with expansive characteristics must be used, composite design should be used with caution and provision must be made in the design to accommodate the expansion.

Composite sections should preferably be proportioned so that the neutral axis lays below the top surface of the steel beam. If concrete is on the tension side of the neutral axis, it shall not be considered in computing moments of inertia or resisting moments except for deflection calculations. Mechanical anchorages shall be provided to tie the sections together and to develop stresses on the plane joining the concrete and the steel.

The steel beams, especially if not supported by intermediate falsework shall be investigated for stability during the time the concrete is in place and before it has hardened.

1.9.2. Shear Connectors

The mechanical means which are used at the junction of the beam and the slab for the purpose of developing the shear resistance necessary to produce composite action shall conform to the specifications of the respective materials as provided in Division II (A.A.S.H.O., Standard Specifications for Highway Bridges, 1961) and in the material specifications (Chapter 4) as set forth in this thesis. The shear connectors shall be of types which permit a thorough compaction of the concrete in order to insure that their entire surfaces are in contact with the concrete. They shall be capable of resisting both horizontal and vertical movement between the concrete and steel.

(A) Metal Shear Connector

The shear connectors shall be riveted or welded to the beams. The capacity of the rivets or welds at permissible working stresses shall equal or exceed the resistance value "Q" of the shear connector.

(B) Epoxy Bonded Aggregate Shear Connector

The maximum shear stresses developed at the epoxy-steel interface shall not exceed the maximum allowable epoxy shear stress as given in Article 1.9.5.(B).

The epoxy bonded aggregate system shall be applied over the entire flange width of the rolled steel section. The aggregate shall be applied utilizing 100 percent coverage.

1.9.3. Effective Flange Width

In composite beam construction the assumed effective width of the slab as a T-beam flange shall not exceed the following:

(1) One-fourth of the span length of the beam.
(2) The distance center to center of beams.
(3) Twelve times the least thickness of the slab.

For beams having a flange on one side only, the effective flange width shall not exceed one-twelfth of the span length of the beam, nor six times the thickness of the slab, nor one-half the distance center to center of the next beam.

1.9.4. Stresses

Maximum compressive and tensile stresses in beams which are not provided with temporary supports during the placing of the permanent dead load, shall be the sum of the stresses produced by the dead loads acting on the steel beams or girders alone and the stresses produced by the superimposed loads acting on the composite beam. When beams are provided with effective intermediate supports which are kept in place until the concrete has attained 75 percent of its required 28-day strength, the dead and live load stresses shall be computed on the basis of the composite section.

In continuous spans, the positive moment portions may be designed with composite sections as in simple spans. The negative moment portions shall be designed on the assumption that concrete on the tension side of the neutral axis is not effective except as a device to develop the reinforcement steel embedded in it. In case reinforcement steel embedded in the concrete is not used in computing the section, shear connectors need not be provided in these portions of the spans.

1.9.5. Shear

(A) Metal Shear Connectors

Resistance to horizontal shear shall be provided by mechanical shear connectors at the junction of the concrete slab and the steel beam or girder. The shear connectors may be continuous helical- or serpentine-shaped bars adequately attached to the beam at regular or variable intervals, or other mechanical devices placed transversely across the flange of the beam, spaced at regular or variable intervals.

The horizontal shear to be transferred by the shear connectors shall be computed by the formula

$S = \frac{Vm}{I}$, in which

S = the horizontal shear per linear inch at the junction of the slab and beam at the point in the span under consideration.

V = the total external shear due to the superimposed loads applied after the concrete has attained 75 percent of its required 28-day strength.*

m = the statical moment of the transformed compressive concrete area about the neutral axis of the composite section or the statical moment of the area of reinforcement embedded in the concrete for negative moment.

I = the moment of inertia of the transformed composite beam.

In the above, the compressive concrete area is transformed into an equivalent area of steel by dividing the effective concrete flange width by the modular ratio "n".

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* For beams erected without temporary shoring, the total external shear "V" is the total external shear from live load and impact plus any shear from dead load added after the concrete has attained the 75 percent of its required 28-day strength. For beams provided with properly designed temporary shoring during construction, "V" is the external shear from dead, live, impact and shoring-removal loads.

The resistance value "Q" at working load of an individual shear connector, or of one pitch of continuous bar, is its critical load capacity divided by the factor of safety. The required pitch of the shear connectors is determined by dividing the resistance "Q" of all connectors at one transverse beam cross section by the shear "S" per linear inch with a maximum pitch of 24 inches.

The useful capacities of the following types of shear connectors, as determined by tests, are as follows:

Channels--$Q_{uc} = 180\ (h + \frac{1}{2}t)\ w \sqrt{f'_c}$

Welded studs (for ratios, H/d equal or greater than 4.2)

$$Q_{uc} = 330\ d^2 \sqrt{f'_c}$$

Helical bars (spirals)--$Q_{uc} = 3840\ d \sqrt[4]{f'_c}$

In the above the following notations apply:

Q_{uc} = the critical load capacity of one shear connector or one pitch of a spiral bar.

h = the maximum thickness of a channel flange, in inches, measured at the face of the web.

t = the thickness of the web of a channel shear connector, in inches.

w = the length of a channel shear connector, in inches, measured in a transverse direction on the flange of a beam.

f'_c = the required compressive strength of concrete at 28 days, as determined by tests of 6" x 12" cylinders, in pounds per square inch.

d = diameter of studs or of the round bars used in helical connectors (spirals), in inches.

H = height of stud, in inches.

The factors of safety to be used shall be determined as follows:

$$*F.S = \frac{2.7\ (1 + C_{mc} + C_{mi}\ C_s) - (C_{mc} + C_{mi}) + C_v}{(1 + C_v)}$$

* A factor of safety of 4 may be used in lieu of calculating the factor of safety by these formulas.

where

$$C_{mc} = \frac{\text{Max, moment caused by dead loads acting on the composite section}}{\text{Max. moment caused by live load}}$$

$$C_{mi} = \frac{\text{Max. moment caused by dead loads acting on the steel beam alone}}{\text{Max. moment caused by live load}}$$

$$C_s = \frac{\dfrac{\text{Moment of inertia of composite beam at point of max. moment}}{\text{Distance from neutral axis to extreme tensile fiber}}}{\dfrac{\text{Moment of inertia of steel beam at point of max. moment}}{\text{Distance from neutral axis to extreme tensile fiber}}}$$

$$C_v = \frac{\text{Vertical shear at section considered caused by the dead load acting on the composite section}}{\text{Vertical shear caused by live load}}$$

The intensity of unit shearing stress in a composite beam may be determined on the basis that the web of the steel beam carries the total external shear, neglecting the effects of the steel flanges and of the concrete slab. The shear may be assumed to be uniformy distributed throughout the gross area of the web.

(B) Epoxy Bonded Aggregate Shear Connector

Epoxy bonded aggregates, when used as a shear connector in composite construction, shall conform to the specifications as set forth in Article 1.9.2. The horizontal shear stress to be transferred by the epoxy bonded aggregate system shall be computed by the following formula

$$S = \frac{Vm}{Ib},$$ in which

S = the horizontal shear stress produced at the junction of the slab and beam.

V = the maximum external shear force due to the superimposed loads applied after the concrete has attained 75 per cent of its required 28-day strength.*

m = the statical moment of the transformed compressive concrete area about the neutral axis of the composite section or the statical moment of the area of reinforcement embedded in the concrete for negative moment.

I = the moment of inertia of the transformed composite beam.

b = flange width of rolled steel section.

In the above, the compressive concrete area is transformed into an equivalent area of steel by dividing the effective concrete flange width by the modular ratio "n".

The maximum horizontal shear stress (S) as determined from the above formula shall not exceed a maximum allowable epoxy shear stress of 200psi. This allowable shear stress of 200psi shall be applicable to beams with either simple or continuous supporting conditions.

The intensity of unit shearing stress in a composite beam may be determined on the basis that the web of the steel beam carries the total external shear, neglecting the effects of the steel flanges and of the concrete slab. The shear may be assumed to be uniformly distributed throughout the gross area of the web.

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* For beams erected without temporary shoring, the total external shear "V" is the total external shear from live load and impact plus any shear from dead load added after the concrete has attained 75 per cent of its required 28-day strength. For beams provided with properly designed temporary shoring during construction, "V" is the external shear from dead, live, impact and shoring-removal loads.

1.9.6. Deflections

The provisions of Article 1.6.10 (A.A.S.H.O., Standard Specifications for Highway Bridges, 1961) in regard to deflections from live load plus impact shall be applicable also to composite beams and girders. Preferably the ratio ofthe length of span to the over-all depth of beam(concrete slab plus steel beam or girder) shall not be greater than 25 and the ratio of the length of span to the depth of the steel beam alone shall be not greater than 30. For continuous spans the span length shall be considered as the distance between dead load points of contraflexure.

When the beams are not provided with falsework or other effective intermediate support during the placing of the concrete slab, the deflection due to the weight of the slab and other permanent dead loads added before the concrete has attained 75 per cent of its required 28-day strength shall be computed on the basis of no composite action.

5.3. Commentary on Design Specifications

1.9.1. General

No changes or additions were made in this article.

1.9.2. Shear Connectors

This article has been modified to include the various design provisions which are necessary in the designing of an epoxy bonded aggregate shear connector. Direct reference has been made to the material specifications concerning epoxy bonded aggregates. These material specifications are shown in chapter 4 and indicate all necessary controls placed on the materials comprising the epoxy bonded aggregate system. As can be seen in the article the shear connectors have been classified into two types. The metal shear connectors include all of the connectors which are welded to the flanges of the rolled steel sections. The specifications for these metal connectors have not been changed. Sub-article B has been added to include provisions for the designing of any epoxy bonded aggregate shear connector. Utilization of 100 per cent aggregate coverage indicates the maximum amount of aggregate which can be randomly placed in the fresh epoxy.

1.9.3. Effective Flange Width

No changes or additions were made in this article.

1.9.4. Stresses

No changes or additions were made in this article.

1.9.5. Shear

This article has been divided into two sub-articles to distinguish between the two types of shear connectors to be utilized in composite construction. As can be seen, the specifications pertaining to the metal or welded shear connectors has not been changed. The addition of sub-article B was made to establish the maximum design shear stress to which the epoxy bonded aggregate shear connector may be subjected. Comparison of this maximum design shear stress with the ultimate shear stresses obtained in the T-Beam tests (Chapter 3) indicates that a factor of safety of 6.51 - 11.20 will be obtained when an epoxy bonded aggregate system as specified in this thesis is utilized.

1.9.6. Deflection

No changes or additions were made in this article.

An example design problem utilizing an epoxy bonded aggregate shear connector has been presented in appendix B. This example problem is a preliminary design for a composite bridge with a simple span length of 85 feet and utilizes A.A.S.H.O. specifications. The design of the slab and supporting members has been included to illustrate that the design procedure is exactly the same for composite beams utilizing epoxy bonded aggregate shear connectors when compared to composite beams with welded shear connectors. The only difference which exists is in the actual design of the shear connector itself.

CHAPTER 6

CONSTRUCTION CONSIDERATIONS

6.1. Introduction

The following suggestions are given to aid in the development of effective application and construction techniques concerning the use of epoxy bonded aggregates as a shear connector in composite construction.

6.2. Surface Preparation

All steel contact surfaces must be sandblasted thoroughly to remove all mill scale and rust. The surface should then be cleaned with methylethyl ketone or grain alcohol to remove any oil or dust particles left by the sandblasting process. It is suggested that only the two cleaning solvents as stated above be used since they do not leave a film on the surface after evaporation. Proper cleaning of the contact surfaces is an absolute necessity and, therefore, great care should be taken with this cleaning process. Solvent film, rust, oil, and dust remaining on the contact surfaces greatly inhibit the bonding ability of the epoxy and must be avoided in all cases.

6.3. Form Work for Epoxy Layer

In order to maintain the proper epoxy bond line thickness, it is suggested that some sort of formwork be placed on the outside edges of the flange or contact surface. This formwork should be constructed from a polyethylene material or any other material covered with a polyethlene film. The form should be placed in such a manner so as to extend above the contact surface a height equal to the required epoxy thickness. This facilitates the application of the epoxy formulation.

6.4. Calculation of Epoxy Volume

The amount of epoxy resin required for a beam and/or beams may be determined by simply calculating the volume of the epoxy layer; that is, the volume enclosed within the formwork. It is suggested that an increase in volume of 5-10 percent be added to this in order to allow for spillage losses and preparation of control specimens.

6.5. Epoxy Formulation

To obtain a more reliable mix the three component system, as shown on page 16 of this thesis, is to be utilized. The proper amount of each constituent should be weighed out as accurately as possible; these weights being determined on the basis of absolute volume.

6.6. Mixing

Parts A and B are added to each other and thoroughly mixed by mechanical or manual means. After this mix is completed, Part C is added and thoroughly stirred into the mixture. It is suggested that a mechanical means of mixing be employed because the final mixture is of a very viscous nature. However, the mixing speed should be controlled in such a manner so as not to allow an excessive amount of air entrainment or formulation of bubbles in the mixture. Mixing in all cases should continue until a mixture of uniform color and texture is obtained.

6.7. Application of Epoxy Formulation to Contact Surfaces

The epoxy formulation is applied to the contact surfaces by use of trowels and spatulas. The formulation should be spread as quickly as possible; once the formulation starts to set up it is very difficult to work with and usually does not develop proper bond. The pot life of the formulation varies for different sized batches; however, a pot life of approximately 30 minutes can usually be expected. Extreme heating of the epoxy denotes the end of its useability. The quality control specimens should also be prepared at this time.

6.8. Placing of Aggregate

The aggregate is immediately hand placed into the fresh epoxy. This must be done before the epoxy starts to harden or set up, otherwise proper bond will not be developed between the aggregate and epoxy. Once the aggregate has been placed in the fresh epoxy it should be pushed down slightly to insure proper bonding. Care must be taken so as not to force the aggregate to come into contact with the steel surface. It is thought that when the concrete is placed, the aggregate will transmit water to the steel, thus causing a rust spot to develop.

6.9. Curing

Once the epoxy formulation has taken its initial set the concrete may be placed. However, in no case should the concrete be placed until the control specimens for the epoxy formulation have been tested. When time is not a factor in construction, it is suggested that the epoxy bonded aggregate system be allowed to cure for at least four days at or above a room temperature of $72^{\circ} \pm 5^{\circ}F$. In construction where it is necessary to place the concrete as soon as possible, accelerated curing of the epoxy control specimens may be utilized. This may be done by placing the specimens in an oven at a temperature of $200^{\circ}F$ for no less than three hours. If the control specimens indicate that the epoxy has not developed the required strength, the surface may be recleaned and a new system applied.

6.10. Note of Caution

The use of rubber gloves and protective clothing is necessary in areas where Epi-Rez 510 and converters are handled and used. Areas of the body which come in contact with the resin and/or converters should be cleaned with denatured alcohol and then be well scrubbed with soap and water.

REFERENCES

1. Beedle, Blackmon, Cooper, Driscoll, Eney, Errera, Estuar, Fisher, Galambos, Hansell, Lay, Levi, Lu, Ostapenko, Reemsnyder, Rumpf, Slutter, Tall, Yen, Structural Steel Design, The Ronald Press Company, New York, 1964, chapter 13, pg. 419.

2. Beedle, Blackmon, Cooper, Driscoll, Eney, Errera, Estuar, Fisher, Galambos, Hansell, Lay, Levi, Lu, Ostapenko, Reemsnyder, Rumpf, Slutter, Tall, Yen, Structural Steel Design, The Ronald Press Company, New York, 1964, chapter 13, pg. 420.

3. Beedle, Blackmon, Cooper, Driscoll, Eney, Errera, Estuar, Fisher, Galambos, Hansell, Lay, Levi, Lu, Ostapenko, Reemsnyder, Rumpf, Slutter, Tall, Yen, Structural Steel Design, The Ronald Press Company, New York, 1964, chapter 13, pg. 431.

4. Miklofsky, H.A., Gonsior, M.J., Santini, J.J., "Further Studies of Epoxy Bonding Compounds," Rensselaer Polytechnic Institute, Physical Research Project No. 13, December, 1963, pg. 1.

5. Kriegh, James D., Endebrock, Elton G., "The Use of Epoxy Resins in Reinforced Concrete-Static Load Tests, Part I," The University of Arizona, January, 1962.

6. Kriegh, James D., Endebrock, Elton G., "The Use of Epoxy Resins in Reinforced Concrete-Static Load Tests, Part II," The University of Arizona, January, 1963.

7. Kriegh, James D., Endebrock, Elton G., "The Use of Epoxy Resins in Reinforced Concrete-Static Load Tests, Part II," The University of Arizona, January, 1963.

8. Kriegh, James D., Endebrock, Elton G., "A Study of the Adhesive and Material Properties of an Epoxy Resin Formulation as a Function of Proportioning the Ingredients, Part I and Part II," The University of Arizona, July, 1963.

9. Kriegh, James D., Endebrock, Elton G., "A Study of the Adhesive and Material Properties of an Epoxy Resin Formulation as a Function of Proportioning the Ingredients, Part I and Part II," The University of Arizona, July, 1963.

REFERENCES--Continued

10. Kriegh, James D., Endebrock, Elton G., Hoopes, Jay N., "A Study to Determine the Aging Effects on an Epoxy Formulation," The University of Arizona, January, 1964.

11. Kriegh, James D., Endebrock, Elton G., "The Use of Epoxy Resins in Reinforced Concrete-Dynamic Load Tests," The University of Arizona, August, 1963.

12. Kriegh, James D., Endebrock, Elton., "Static Test of an Epoxy Joined Composite T-Beam at a Low Temperature, " The University of Arizona, September, 1964.

APPENDIX A

CONCRETE MIX DESIGN

(OVEN DRY BASIS)

BASE MIX

CEMENT	94 LBS
SAND	160 LBS
COARSE AGGREGATE	229 LBS
WATER	5.23 GAL

WATER

IN SAND (6.5%)	10.40 LBS
IN AGGREGATE (1%)	2.29 LBS
TOTAL	12.69 LBS
TOTAL	1.52 GAL

FIELD MIX

CEMENT	94 LBS
SAND	170 LBS
COARSE AGGREGATE	231 LBS
WATER	3.71 GAL

CLASS OF CONCRETE	"S"
SLUMP	2"
W/C RATIO	4.53 G.P.S.
CEMENT PER CU. YD.	7.50 SACKS
% OF AIR ENTRAINED	4.0 %
CEMENT	TYPE III

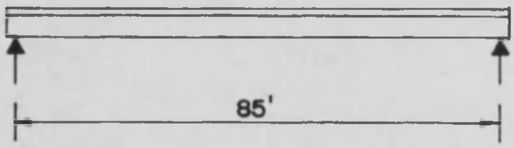
APPENDIX B

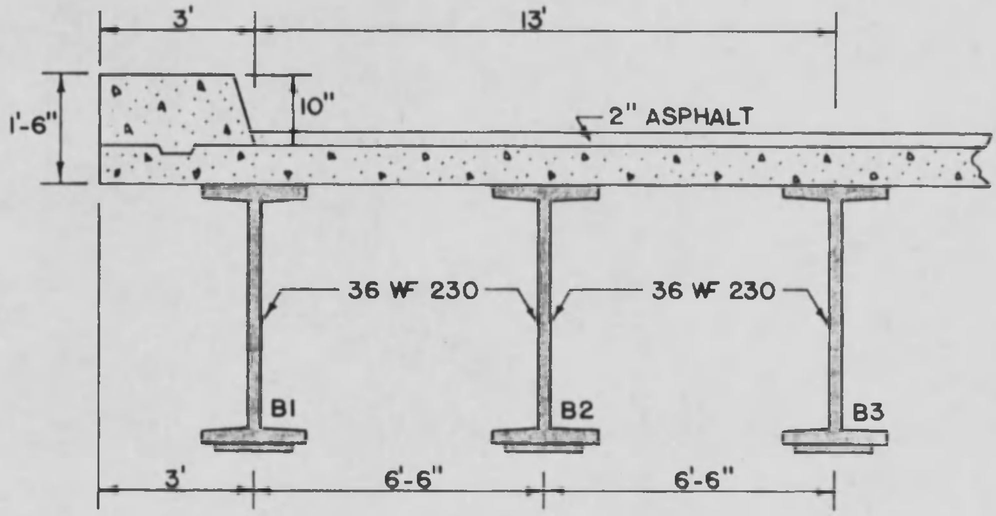
EXAMPLE DESIGN

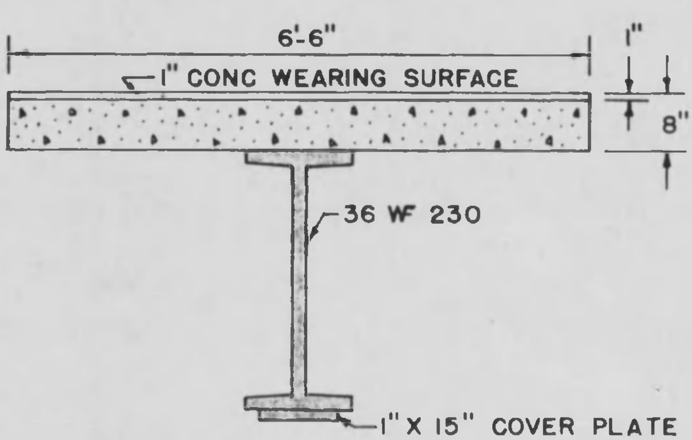
(DESIGN EXAMPLE TAKEN FROM STRUCTURAL STEEL DESIGN, EXAMPLE 13.2 SEE REFERENCE NO. 1)

PROBLEM:

DESIGN A 85' SIMPLE SPAN HIGHWAY BRIDGE FOR H20-S16 LOADING.







APPENDIX B CONT.

SOLUTION:

DEAD LOAD ON STEEL BEAMS

SLAB	6.5(100) =	650 LB/FT
BEAM		280 LB/FT
		930 LB/FT

DEAD LOAD ON COMP. SECTION

WEARING SURFACE	25(6.5) =	163 LB/FT
CURB & RAIL	450 ÷ 2.5 =	180 LB/FT
		343 LB/FT

LOADING: MEMBER B2

7" SLAB	=	85 PSF
2" ASPHALT	=	25 PSF
RAILING	=	75 LB/FT
L. L.	=	H20-S16
CURB	=	375 LB/FT
1" CONC. W. S.	=	12.5 PSF

ROADWAY SLAB

BEAM SPACING = 6'-6" O.C.

S = 78" - 8.25" = 69.75" OR 5.81'

$M_L = \frac{S+2}{32} P = \frac{7.81}{32}(16) = 3.905$ KIP-FT. PER FT. OF SLAB

FOR SLABS CONTINUOUS ON THREE OR MORE SUPPORTS

$M_L = 0.8(3.905)12$	=	37.49	KIP-IN. PER FT. OF SLAB
$M_D = \frac{1}{10}(0.088)(5.81)\ 12$	=	3.57	KIP-IN. PER FT. OF SLAB
$M_I = 0.30(37.49)$	=	11.25	KIP-IN. PER FT. OF SLAB
$M_{WS} = \frac{1}{10}(0.038)(5.81)\ 12$	=	1.54	KIP-IN. PER FT. OF SLAB
TOTAL M	=	53.85	KIP-IN. PER FT. OF SLAB

FOR 7" CONCRETE SLAB $d = 7.00 - 1.50 = 5.50"$

MAIN REINFORCEMENT

TRY #5 BARS AT 6" O.C. $A_s = 0.62$ IN.^{2} PER FT. OF WIDTH

$$j = 1 - \frac{K}{3} = 1 - \frac{0.350}{3} = 0.883$$

APPENDIX B CONT.

$$f_s = \frac{M}{A_s jd} \quad \frac{53,850}{0.62(0.883)5.5} = 17,880 \text{ PSI}$$

$$K = \sqrt{2pn + (pn)^2} - pn = \sqrt{2(0.0094)10 + (0.094)^2} - 0.094 = 0.350$$

$$f_c = \frac{2pf_s}{K} = \frac{2(0.0094)17,880}{0.350} = 961 \text{ PSI}$$

LONGITUDINAL REINFORCEMENT

PERCENTAGE $= \frac{220}{\sqrt{S}} \quad \frac{220}{\sqrt{5.81}} = 91\%$, MAXIMUM = 67 %

P = 0.67 (0.0094) = 0.0063

TRY # 5 BARS AT 8" O.C.

$$P = \frac{0.460}{12(5.5)} = 0.00697$$

LIVE LOAD DISTRIBUTION

$$\frac{S}{5.5} = \frac{6.5}{5.5} = 1.182$$

IMPACT FACTOR

$$I = \frac{50}{125 - L} = \frac{50}{125 - 85} = 0.238$$

	SHEAR		MOMENT	
D.L.	0.930 (42.5)	= 39.5 KIP	$0.930(85)^2 1.5$	= 10110 KIP-IN
W.S.	0.343 (42.5)	= 14.6 KIP	$0.343(85)^2 1.5$	= 3720 KIP-IN
L.L.	1.182 (64.1) 0.5	= 37.9 KIP	1.182 (1254.7) 6	= 8920 KIP-IN
I	37.9 (0.238)	= 9.1 KIP	8920 (0.238)	= 2120 KIP-IN

APPENDIX B CONT.

SUMMARY OF SECTION PROPERTIES

SECTION	LOCATION	STRESS CALCULATION		
		INITIAL D.L.	L.L.+ I	COMP. D.L.
NO COVER PLATE	TOP CONC.		S_c = 1899 IN.^{3}	S'_c = 1060 IN.^{3}
	TOP STEEL	S = 835.5 IN.^{3}	S_t = 3480 IN.^{3}	S'_t = 1612 IN.^{3}
	BOT. STEEL	S = 835.5 IN.^{3}	S_b = 1057 IN.^{3}	S'_b = 963 IN.^{3}
WITH COVER PLATE	TOP CONC.		S_{cp} = 2150 IN.^{3}	S'_{cp} = 1194 IN.^{3}
	TOP STEEL	S_{st} = 900.0 IN.^{3}	S_{tp} = 3460 IN.^{3}	S'_{tp} = 1690 IN.^{3}
	BOT. STEEL	S_{sb} = 1230 IN.^{3}	S_{bp} = 1550 IN.^{3}	S'_{bp} = 1415 IN.^{3}

I = 14,988.4 IN.^{4}, I_s = 19,165.4 IN.^{4}, I_c = 29,111.9 IN.^{4}, I_{cp} = 39,573 IN.^{4}

MAXIMUM BENDING STRESS

	M_D	M_L	M_d		
TOP CONCRETE	—	$-\dfrac{11040}{2150(10)}$	$-\dfrac{3720}{1194(10)}$	= -0.825	KSI
TOP STEEL	$-\dfrac{10110}{900}$	$-\dfrac{11040}{3460}$	$-\dfrac{3720}{1690}$	= -16.74	KSI
BOTTOM STEEL	$+\dfrac{10110}{1230}$	$+\dfrac{11040}{1550}$	$+\dfrac{3720}{1415}$	= 17.99	KSI

COVER PLATE CALCULATION WILL NOT BE SHOWN.

SHEAR CONNECTOR

USE EPOXY BONDED AGGREGATE SHEAR CONNECTOR

ASSUMING COVER PLATE

I = 39,573 IN.^{4}

$M = AY$ = 54.6(14.91) = 816 IN.^{3}

b = 16.475 IN.

V_{L+I} = 47.0 KIP

$$S = \frac{VM}{Ib} = \frac{47000(816)}{39573(16.475)} = 58.8 \text{ PSI} < 200 \text{ PSI}$$