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GREAT LAKES WINTER NAVIGATION-- TECHNICAL AND ECONOMIC ANALYSES

VOL. IV: STRENGTHENING OF STEEL PLATES USING FERROCEMENT AND REINFORCED CONCRETE

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Report to Department of Commerce
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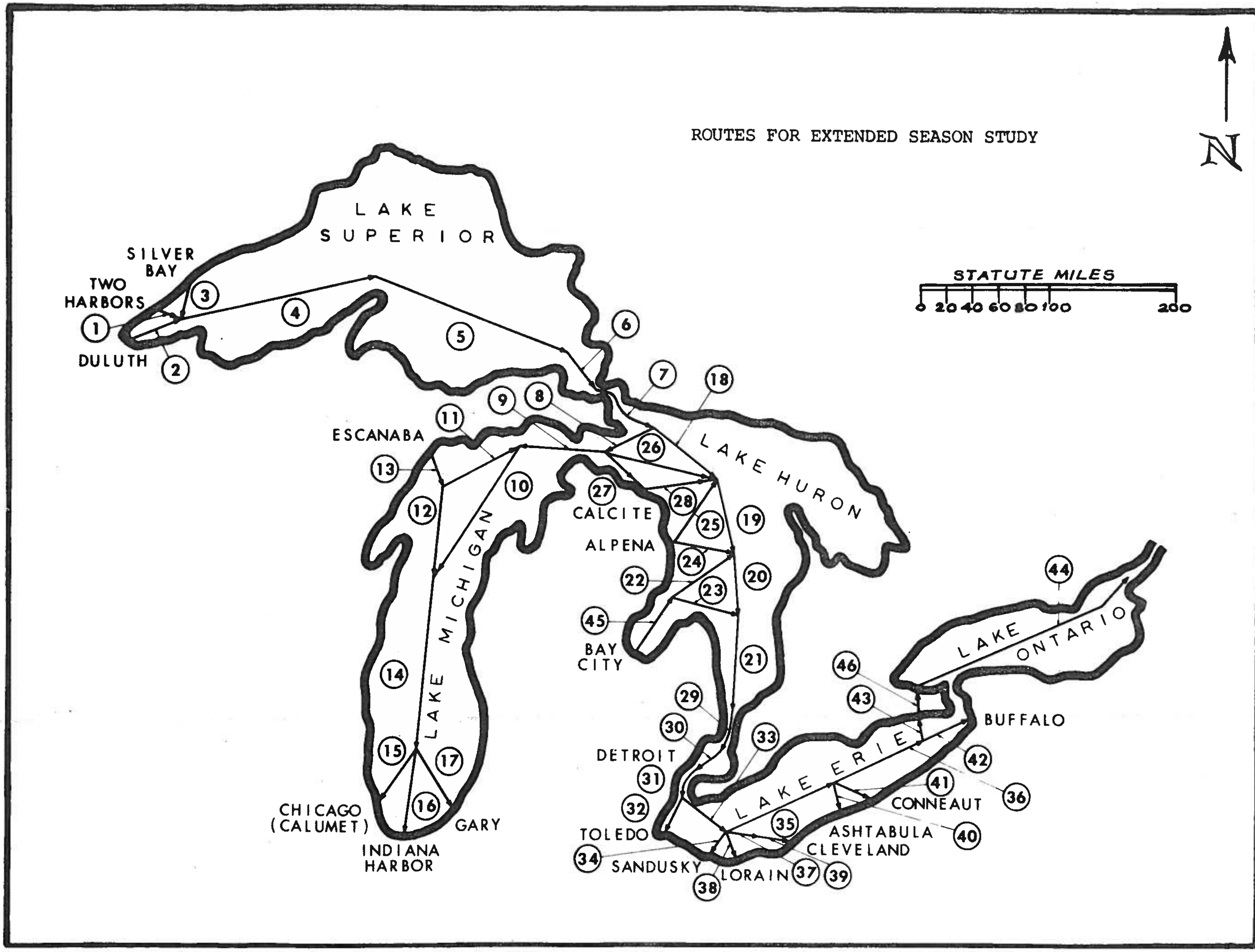
ROUTES FOR EXTENDED SEASON STUDY



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GREAT LAKES WINTER NAVIGATION--TECHNICAL AND ECONOMIC ANALYSES

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Page 19, top line

READS	SHOULD READ
$0.85\ f'_c$	$0.85\ f'_c$ [1]

Page 28, third complete paragraph, line 2

t_R	t_r

Page 29, Equation [9-2a]

$x = \frac{(t-2pdf_y/f_x)t}{(t-wpdf_y/f_s) + t}$	$x = \frac{(t-2pdf_y/f_s)t}{(t-2pdf_y/f_s) + t}$

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ABSTRACT

To determine the applicability of using ferrocement and reinforced concrete for ice strengthening ship hulls, thirty-four composite beams were constructed and tested. Beams were made of 1/4-inch steel plate reinforced with 1 inch of either a concrete or ferrocement segment. Shear transfer between plate and concrete was accomplished using natural bond, epoxy or shear studs. Ferrocement composite beams were found to be strongest but less ductile than those of reinforced concrete. A sandblasted steel surface plus epoxy provided adequate shear transfer although shear studs allowed the greatest ductility. It is concluded that ships may be adequately strengthened using ferrocement and reinforced concrete; a brief design guide is given to aid in selection of an appropriate concrete section.

ACKNOWLEDGMENTS

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1. INTRODUCTION

Waterborne commerce in the Great Lakes is currently limited to an eight or nine month season because of ice during the winter. In order to increase the shipping season to a full year, existing ships would require strengthening to resist ice pressures which may be as great as 600 psi. Conventional reinforcing methods involve adding steel to the shell (in the form of doublers) or to the internals, or both. No matter what combination is used, straight steel reinforcement is invariably expensive. A second method, which has been used in Finland, is to add reinforced concrete or ferrocement to the interior side of the shell. Concrete material and construction costs are less than those of steel, and the unit weight of concrete construction is about one-third that of steel of equal strength. This makes the possibility of using reinforced concrete or ferrocement quite attractive.

Reinforced concrete is composed of concrete and relatively widely spaced steel reinforcing bars. Ferrocement, on the other hand, is a composite made using a mortar matrix with closely-spaced fine steel wire mesh reinforcement. The two materials are nearly the same with regard to overall mechanical properties, but whereas reinforced concrete sections crack visibly at low loads, the mesh reinforcement of ferrocement arrests microcrack propagation; this is important for marine applications, where watertightness is paramount.

The purpose of this study was to investigate reinforced concrete and ferrocement construction as they apply to ships and to determine possible methods of construction and analysis of these materials for the strengthening of vessels. Little is known about the performance of composites of steel plate coupled to reinforced concrete or ferrocement, (which would be the situation for existing ships reinforced against ice with supplementary concrete), so an experimental program was undertaken to provide data on the strength of reinforced concrete, and ferrocement, composite steel construction.

2. PREVIOUS CONCRETE OCEAN STRUCTURES

The history of the use of concrete for ocean structures has been well documented. The Romans used concrete to build a wharf in the first century;that concrete remains in good condition today. Pier and harbor construction using plain and reinforced concrete has shown that concrete performs excellently in the ocean environment. The most severe problems occur at the splash zone during freeze-thaw conditions, and failures of low strength concretes have been caused. Yet in all conditions, if the concrete is of high quality (strength over 5000 psi) and the reinforcement is adequately covered (typically 3 to 4 inches), no deterioration of the structure will occur.

Reinforced concrete ships and barges, up to 2500 tons displacements and 400 ft lengths, were built by Great Britain and the U.S. during both World Wars. Many of the early vessels lasted some 35 years without any sign of deterioration in their 4 to 6 inch thick hulls. More recently, reinforced concrete has been used to construct pontoons for docks in Vietnam as well as for large floating bridges across Lake Washington near Seattle.

The first floating concrete structures were in fact made of ferrocement--Lambott building two ferrocement rowboats between 1848 and 1849, one of which is still in good condition. Pier Luigi Nervi constructed ferrocement crafts during and just after World War II. One of these was a 38-foot ketch which had a 1/2 inch thick hull consisting of 7 layers of steel fabric and 1/4 inch round bars; this ketch is still in good condition. Since the early 1960's many pleasure craft have been constructed using ferrocement. The hulls are generally 3/4 inch to 1 1/2 inches thick; several layers of chicken wire are laid on each side of a 1/4 inch bar frame, and high strength mortar is troweled into the reinforcement.

Despite these constructions in ferrocement, relatively few experimental investigations have been conducted on the material. A summary is given in the next chapter of those few previous studies, which leads us into a description of the experimental program performed by the authors. The major part of the present study is related to the determination of steel hull/concrete reinforcement mechanical behavior for use in subsequent design.

3. PREVIOUS STUDIES ON FERROCEMENT

The general purpose of tests on ferrocement has been to determine if ferrocement is a true composite material which exhibits synergism, i.e., whether the combination of fine steel reinforcement and mortar is stronger (and most particularly, tougher) than the sum of the individual components. The close spacing of the wire mesh does influence cracking strength and increases the ductility of ferrocement beyond that of typical reinforced concrete with small cracks.

Chang, Gibson and Gibbons[4] built 85 ferrocement panels which were tested in flexure. These panels, 5 inches wide and 28 inches long, were built of various thicknesses using Type I and Type II Portland Cement*, fine silica sand, and a water to cement ratio of 0.44. Expanded metal and woven wire mesh were used as reinforcement. The behavior of these ferrocement panels was similar to reinforced concrete beams, in that over-reinforced panels with no compression steel failed by concrete crushing, and under-reinforced panels with compression reinforcement failed when the upper layers of mesh buckled. Cracks appeared at early stages of loading but remained fine until the load was about 85 percent of the ultimate. Standard reinforced concrete theory closely, yet conservatively, predicted the flexural behavior of ferrocement, a fact also brought out in a study by Muhlert[14]. His panel reinforcement samples included 1/4 inch or 3/16 inch bars at the center of the section with equal layers of mesh on either side, which is typical ferrocement boat construction.

Naaman and Shah[15] tested 1/2 inch thick 3-inch by 12-inch ferrocement panels in direct tension to determine cracking and ultimate strengths; the compressive strength of ferrocement is essentially that of the mortar alone. Reinforcement included galvanized woven wire mesh of many sizes as well as chicken wire. A parameter "specific surface", was defined as the ratio of total surface area of longitudinal mesh reinforcement per unit volume of composite, and it was found that the composite stress at initial cracking

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*Portland Cement types designate different chemical mixes designed for various purposes. Type I is for normal, terrestrial use; Type II for water-front and foundation construction where more corrosive conditions are present; Type III is for fast setting; Type IV is for mass concrete structures; Type V is for severe corrosive conditions; and Type K is used to eliminate shrinkage of the concrete.

increased as the specific surface increased. Generally cracking was similar to reinforced concrete except that the transverse reinforcement of ferrocement decreased the longitudinal spacing which is associated with bond failure.

The composite modulus of elasticity after the first crack (E_c) was computed by the simple law of mixtures and was approximately given by the following

$$E_c = E_{RL}P$$

where

E_{RL} = effective modulus of the mesh (psi)

P = ratio of steel to gross area in the loading direction

For 2 percent reinforcement*, $E_c=0.4\times10^6$ psi, which was slightly less than measured values of E_c.

The elongation of the specimens at ultimate load was about equal to that of the mesh alone. The ultimate tensile load capacity of the ferrocement equaled the ultimate capacity of the mesh.

Tancreto and Haynes[20] tested 1 inch thick 9-inch by 30-inch long ferrocement panels loaded in flexure, with a span length of 27 inches and load lines of 1 1/2 inches on each side of center. Panels were reinforced using woven wire mesh, chicken wire or reinforcing bars. Mortar was made using Portland Cement Type II and fine sand with a water-to-cement ratio of 0.7; the ultimate, 28 day mortar strength (f_c') was 6000 psi. The load at the first visible crack was substantially greater for panels with 2 percent reinforcement than for those with 1 percent reinforcement; only a minor load difference was noted between 2 percent and 3 percent. The visible cracking strength ratio (R_{VC}) is given by the following:

$$R_{VC} = 1.93 \log_{10} D - 0.13$$

where

R_{VC} = stress at visible cracking for the ferrocement panel divided by the ultimate flexural stress for a plain mortar panel.

D = wire density, number of longitudinal wires per square inch of cross section.

*Percent reinforcement as defined here and throughout this report is the ratio of the area of longitudinal steel to the gross area of the cross section.

The average secant modulus drawn to the visible cracking load was 1.8×10^6 psi. Fewer visible cracks were seen in panels with fine closely spaced wires than in other panels with equal percentage reinforcement.

The ultimate strength of the panels reinforced with mesh was directly related to the mesh properties. The ultimate moment capacity was given approximately by the following relation:

$$M_u = \frac{F_u I}{c}$$

where

M_u = ultimate moment (in-lbs)

F_u = ultimate wire strength (lbs/in^2)

I = second moment of area of wires about the neutral axis which was assumed at mid-height of sections (in^4)

c = distance from the neutral axis to extreme wire in tension (in)

Tancreto and Haynes suggested that the ultimate design load be limited to 70 percent of the ultimate load calculated using the above formula.

Panels with mesh layers grouped near the top and bottom did not resist higher loads than the panels with evenly distributed layers. For panels with 2 percent reinforcement using 4 X 4 mesh of 0.025-inch diameter wire (i.e., 4 wires per inch in each direction), the ultimate strength was twice that of panels with an equal amount of chicken wire and twice that of panels reinforced with plain steel rods of 0.15-inch diameter. The 4 x 4 mesh with 0.025-inch diameter wire was judged the best reinforcement for ferrocement because of its good mechanical properties, very good workability, and low cost.

In summary, previous research on ferrocement has shown that it may be regarded as a thin reinforced concrete material with high cracking resistance. The latter is particularly valuable for ocean structures; elimination of cracks reduces corrosion of the reinforcement and minimizes leakage.

4. COMPOSITE HULL CONSTRUCTION

In a ship hull reinforced with concrete, it would be desirable to have both steel and concrete act in unison as a composite. This requires an efficient "shear connection" across the hull/concrete interface, otherwise the applied loads and moments will not be shared by the components in the structure. The usual "in situ" steel-concrete bond is not adequate, so shear connection is made with epoxy adhesive, or studs or other projections attached to the steel at the interface. Some information is available for the composite steel/concrete construction seen in buildings and bridges (where the steel I-beams are joined to reinforced concrete decks), but little is known about shear connections between flat steel plate and concrete as envisaged for hull reinforcement. This section summarizes those previous studies on shear connectors for steel/concrete composites.

4.1 Shear Connection with Studs

Many studies have been conducted on the shear connection of steel beams to concrete decks using steel studs. One of the first and most comprehensive was by Viest[21] who conducted direct shear tests of specimens made of steel wide-flange sections connected to concrete slabs with 1/2-inch to 1 1/4-inch diameter studs. "Push-off" tests by Slutter and co-workers[6,17,14] gave a relationship for the ultimate shear load Q_u per stud as

$$Q_u = 1.106\, f_c'^{\,0.3}\, E_c^{\,0.44}\, A_s$$

where

A_s = area of stud (in^2)

f_c' = ultimate (compressive) strength of concrete (psi)

E_c = modulus of elasticity for concrete (psi)

The E_c was approximately that given by the American Concrete Institute (ACI)[1] formula:

$$E_c = 57{,}000\sqrt{f_c'}$$

Thus $Q_u \simeq 140\, f_c'^{\,0.52}\, A_s$, which may be contrasted with an expression from Viest[21]

$$Q_u \simeq 424\, f_c'^{\,0.5}\, A_s$$

It is not surprising that recent tests[17] show that Slutter and Driscoll's work[19] (which forms the basis of the American Institute of Steel Construction current standard) has a safety factor of about two.

The load-slip relation was approximated by the following:

$$Q = Q_u(1 - \exp(-18s))^{0.4}$$

where

Q = load per stud (kips)

s = slip (in)

At a slip of 0.2 inch the load was 99 percent of the ultimate. The load-slip relation for reloading, where the original bond has been destroyed was given by

$$Q = Q_u\left(\frac{80s}{1 + 80s}\right)$$

Some studies have been performed on composite construction using steel plates connected to concrete[3,10,18]. Casillas, Khachaturian and Siess[3] tested composite beams 12-inches wide, 9 1/4 inches deep with an 8 foot span. Steel plates 1/4 inch thick were used as tension reinforcement at the bottom of the beams, and a few tests included plates used in compression. Wire fabric welded to the plate and Nelson studs of 1/2, 5/8 and 3/4 inch diameter welded to the plate acted as shear connectors. Wire fabric spot welded to the plate was found to be an inadequate shear connector. Complete welding of the fabric was necessary - an uneconomical process. Four studs per square foot was considered best to achieve full composite action. When less than four studs per square foot were used, the typical failure mode of the beams was a bond-shear failure in that excessive slip between the plate and concrete lead to a diagonal shear crack. When the recommended number of studs were used, the plate yielded, and the beam failed in a typical flexural mode. The horizontal shear was determined using the formula:

$$S = \frac{Va}{jd}$$

where

S = total horizontal shear force (lbs)

V = vertical shear force (lbs)

a = shear span (in)

j = ratio of distance between centroid of compression and centroid of tension to depth d

d = depth from compression face of beam to centroid of longitudinal tensile reinforcement

The failure load per stud calculated using this formula gave critical loads which agreed with those predicted using the formula by Viest.[21] It was further determined that if studs were distributed according to the shear diagram the ultimate capacity of the beam was greater than for uniform stud spacing. Some tests were conducted where the plate was loaded in compression and two #7 bars* were used as tensile reinforcement in the concrete. The plate with no shear connectors was considered ineffective in compression.

Tests conducted by Perry, Burns and Thompson[18], which were similar to those by Casillas[3], considered the plate active in compression as well as in tension. As loading was applied to beams with plates in tension, cracks started vertically at or very close to the studs; the crack continued vertically until it reached the top of the stud, then inclined toward the load. Static failure occurred in the steel-concrete boundary layer after considerable shear slip at the interface. Dynamically loaded beams failed in diagonal shear when stirrups were not included.

All beams with the plate in compression had closely spaced stirrups. As loading was applied, vertical cracks developed rapidly to about 1/2-inch below the plate. The beam failed when yielding of tensile reinforcement produced concrete crushing below the plate.

Perry, Burns and Thompson[18] concluded that the plate acted to carry either direct tensile or compressive forces and that any combination of studs could be used as long as their capacities satisfied the forces determined by the shear diagram. Beams with plates in compression exhibited great ductility. As a design guide, zero slip at the plate-concrete joint was recommended for composite sections under repeated loading.

Other tests[8,10], as well as those reported above, lead to the general conclusion that stud shear connectors are an effective way to force composite action between the steel and concrete.

4.2 Shear Connection with Epoxy

The most applicable research on using epoxy as a shear connector in composite construction were tests conducted by Miklofsky and Gonsoir.[13]

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*7/8 in diameter

Epoxy joints were tested in tension, in shear adhesion between steel and mortar, and in tension adhesion between steel and mortar. The shear adhesion specimens were made by painting an epoxy on a steel strap, then immediately casting a mortar block on top. A four day moist room cure, at various temperatures was followed. Some findings include, (1) merely cleaning the steel was not adequate, roughening the surface by means such as sandblasting was necessary to provide a good bond surface; (2) freeze-thaw tests had an adverse effect on the failure strength of shear-bond specimens, with all failures occurring in the mortar so that epoxy deterioration was not determined; (3) tests of shear specimens without epoxy but with the steel sandblasted showed that these specimens had shear resistance less than one-fourth of those with the epoxy. The most significant result was that the shear and tension adhesion specimens failed primarily in the mortar.

Miklofsky and Gonsoir[13] also tested a series of quarter scale composite beams made using epoxy as the shear connector. The tests were compared to ones of concrete T-beams and to composite sections made using stud shear connectors; the results showed that the epoxy provided a satisfactory shear connection.

Precast, hardened concrete beams have been strengthened by epoxy bonding steel plates to their bottoms and sides.[7] Tests showed that the beams acted as composites with the epoxy working well as a shear connector. Further experiments were conducted on beams made by first forming a steel gutter of the bottom and side plates, painting the gutter with epoxy, then casting concrete in the gutter. No forms were necessary; no reinforcing bars were included. The beams showed good static behavior, the epoxy shear connection being adequate.

Kajfasz[11] also tested precast concrete beams to which steel plates and reinforcing bars were epoxy bonded. His principal finding was that the epoxy bond ensured full composite action; the bond was as good as in traditional reinforced concrete structures. Steel plates were considered better reinforcement than the bars because the tensile stress between the steel and concrete was less with the plates.

The few studies described above indicate that an epoxy bond will provide an adequate shear connection if the steel surface is properly prepared.

5. EXPERIMENTAL PROGRAM

5.1 Experimental Design

The experimental program was designed to determine the composite behavior of beams made using steel plate and thin sections of reinforced concrete or ferrocement. Beams spanning one direction were studied rather than plates in order to simplify testing procedures; in reality the flexural action of a hull may also be considered as one-directional since the span in one direction is typically much larger than the other.

The experiments were designed to investigate techniques to reinforce ship hulls using ferrocement, reinforced concrete and bonding materials. The beam size chosen is approximately a 1/2 to 1/3 scale model of the of the expected, reinforced hull configuration. Also, the results by other researchers were used to plan these tests. Woven wire mesh 4 x 4 x 0.025-inch was selected as ferrocement reinforcement, and a span length of 27 inches and concrete section thickness of one inch were used so that results by Tancreto and Haynes could be easily compared with the test findings. Ungalvanized mesh was used in the ferrocement in order to avoid electrolysis between the zinc galvanizing and steel in the basic concrete environment.

Figure 1 illustrates the various tests which were conducted; three tests of each type were performed. Individual tests of the steel plate, reinforced concrete and ferrocement provided a basis for determining the extent of composite action in the beams made of two materials. A combination of reinforcing bars plus mesh was also included in one series to determine if the favorable properties of both ferrocment and reinforced concrete could be combined in one specimen. Because of the large amount of yielding in the 1/4-inch mild steel reinforcing bars, some specimens were constructed with 3/8-inch Grade 60 steel bars that had deformations to prevent slip between the steel and concrete. Three methods for shear connection between the steel plate and concrete segment were used: (1) steel-to-concrete natural bond (also termed no-bond) [Specimen #4], (2) epoxy adhesive [Specimens #5,6,11-15] and (3) studs of 1/8-inch and 3/16-inch diameter [Specimens #7, 8]. In all cases the steel plates were cleaned of all rust and grease and in several specimens the plates were roughened by sandblasting.

Tests were conducted with steel plate in tension as well as in compression. While the latter condition represents the typical action of a hull reinforced with concrete on the interior, the plate-in-tension tests represent negative moment locations such as those occuring at stiffening members (frames). Also, since a ship operating in heavy ice would receive numerous impacts from the ice, several specimens were tested under cyclic load applications to determine the effect of progressive loading and unloading on bond deterioration and strength failure.

5.2 Specimen Fabrication

All specimens were 12 inches wide and 30 inches long; their section was composed of a one-inch thick concrete segment with 1/4 inch thick cold rolled steel plate. For reinforced concrete sections, 1/4-inch diameter plain and 3/8-inch diameter deformed hot-rolled steel bars were used (called respectively #2 and #3 bars). The ferrocement reinforcement, as mentioned earlier, was ungalvanized 4 x 4 woven wire mesh with wire diameter of 0.025 inch. In most of the concrete segments, 2 percent reinforcement was used: five #2 bars for the reinforced concrete and ten evenly spaced layers of mesh for the ferrocement (Figures 2 and 3). Figure 4 shows a specimen with 2 percent plain steel bars plus 5 layers of mesh (1 percent). Figure 5 shows a specimen with 3.7 percent of Grade 60 deformed steel bars.

The epoxy used for shear connection was Sika brand Colma Fix epoxy which is specifically designed to bond while fluid to wet concrete. After the two-part epoxy was mixed, it was brushed onto the steel plate; the application life of the epoxy was about 30 minutes. For specimens 7 and 8, 98 3/16-inch and 118 1/8-inch studs per beam, respectively, were used as shear connectors; the studs were 7/8-inch long, cut from cold rolled steel rod. Holes of 0.120-inch diameter were drilled into the plate for the 1/8-inch studs and 0.180-inch diameter, for the 3/16 studs. The layout patterns are shown in Figure 6. The studs were hammered into the holes so that 5/8-inch remained protruding. This extension satisfied the requirement that the stud length be four times greater than its diameter. The very tight fit of the studs in the drilled holes is believed to have provided sufficient tensile resistance against pull-out. Figure 7 shows these specimens with reinforcing bars and mesh placed and ready for concrete.

The beams were cast in wooden forms 12 inches wide, 30 inches long and 1 1/4 inches deep. In casting reinforced concrete composite sections, the cleaned plate was laid at the bottom of the form and the five 1/4-inch bars were located in guides at the ends of the form; the mortar was poured, and the form externally vibrated. In casting ferrocement composite sections, the cleaned plate was laid at the bottom of the form and two layers of mesh were placed on top; mortar was troweled into the mesh and vibrated by holding a spud vibrator on top of the mesh (see Figure 8). Five more layers of mesh were laid, and concrete was troweled and vibrated into the mesh. The remaining three layers of mesh were laid and were held in place by placing wires across the top of the form; concrete was troweled and vibrated. When epoxy was used, the epoxy was painted on the plate immediately before it was placed in the form; the mortar was cast while the epoxy was fluid, typically within five minutes after painting (see Figure 9). For non-composite specimens without a steel plate, a plastic sheet was placed between the plate and the concrete; in this manner the concrete segment was insured the correct thickness.

The specimens were removed from the form after one day and cured in a controlled environment, 100 percent humidity at 73°F, 14 days. The specimens were then stored under room conditions for another 14 days until testing on the 28th day after casting.

Six 4 x 8-inch concrete control cylinders were cast with each set of specimens; the cylinders were cured for 28 days at 100 percent humidity. Three of the six cylinders were tested at 28 days to determine the compressive concrete strength (f'_c), the other three cylinders were either compression tested at seven days to determine strength gain of the mortar or were subjected to a split-cylinder test at 28 days to determine the tensile strength of the mortar (Table 1).

The mortar was made of Portland Cement Type I, sand and water mixed according to the following weight ratios: fine aggregate-to-cement ratio of 2.6 and water-to-cement ratio of 0.5. Course aggregate was not used because of the small mesh openings in the ferrocement. The design concrete strength was 6000 psi (Table 1).

Four different kinds of steel were used in constructing the specimens. The steel plate had a yield strength of 32.9 ksi, an ultimate strength of 44.5 ksi, and an elongation at fracture of 35 percent on 2-inch gauge length (Figure 10). Wire mesh used in the ferrocement did not have a marked yield point and had an ultimate strength of 184.2 ksi with an elognation at fracture of 1.1 percent on 6-inch length (Figure 11). The plain steel 1/4-inch diameter reinforcing bars had a yield strength of 51.1 ksi, an ultimate strength of

73.0 ksi, and an elongation at fracture of 25.7 percent (Figure 12). Number 3 (3/8 inch diameter) Grade 60 deformed bars used for Specimens 12 and 13 had a yield strength of 69.6 ksi, an ultimate strength of 101.2 ksi and an elongation at fracture of about 9.5 percent (Figure 13).

6. TEST RESULTS

All specimens were tested as beams, with a span of 27-inches and centrally loaded along a line across the beam width as shown in Fig. 14. A universal testing machine was used to apply the load, and two dial gauges located at the center of the specimen provided the deflection measurements (Fig. 15).

Test results for each series of specimens are presented as load deflection curves which represent average values of three tests. These curves are presented and discussed in the remainder of this chapter. A summary of test results is given in Table 2.

6.1 Steel Plate Alone

Two steel plates without concrete or ferrocement were tested to give a standard for determining the actual composite effect of the constituent portions. The load-deflection curves averaged for these two plates is shown in Figure 16.

6.2 Beams With No Steel Plate

6.2.1 Ferrocement

The load-deflection curve for ferrocement Specimens 2 tested with no plate is shown in Figure 17. At ultimate load the mesh wires fractured one at a time leading to a zipper action mode of failure.

6.2.2 Reinforced Concrete

The load-deflection curve for reinforced concrete Specimens 3 tested with no plate is also shown in Figure 17. The beams' average ultimate load was 970 pounds with a minimum load of 740 pounds and a maximum load of 1212 pounds. The average deflection at ultimate load was 0.91 inches. Failure was gradual with a number of cracks opening up and finally culminating in a very large crack at the center.

6.3 Beams With Steel Plate-in-Compression

6.3.1 Reinforced Concrete with Natural Bond

Specimens 4 were cast on the steel plate, cleaned with acetone, with no provision to bond the concrete to the plate. The load-deflection curve shown in Figure 18, indicates an average ultimate strength of 2080 pounds at an average deflection of 0.97 inches. Failure was gradual with cracks opening up as loading increased. At ultimate load a longitudinal crack had opened up between the concrete and plate reducing the bond strength to zero. Also, a large flexural crack had opened up at the centerline under the load. Concrete crushing adjacent to the plate was observed.

6.3.2 Reinforced Concrete with Epoxy Bond

The load-deflection curve for Specimens 6 is given in Figure 19. The average ultimate load of 2150 pounds occurred at an average deflection of 1.21 inches. A sudden drop in load occurred at a deflection of about 0.22 inches and at a load about equal to the ultimate load of Specimens 4 with the natural bond. This drop in load is associated with bond failure between the concrete and the steel plate. After the bond failure additional displacement produced increased loads until the ultimate load was achieved. After the ultimate load was achieved the beams' loads fell off gradually as the reinforcement yielded.

6.3.3 Reinforced Concrete with Shear Studs

The load deflection curve for Specimen 8 is shown in Figure 20. This system was flexible enough so that a distinct ultimate load was not achieved--the load continued to increase slightly with continued deflection. The test was terminated because of excessive deflection. For all three specimens deflections exceeding two inches were recorded without a drop in load capacity. The average recorded maximum load was 2460 pounds.

6.3.4 Ferrocement with Epoxy Bond

The load-deflection curve for ferrocement beam Specimen 5 is also shown in Figure 19. The average ultimate load of this specimen was 4040 pounds which occurred at an average deflection of 0.70 inches. The ultimate loads and deflections were uniform with a range from maximum to minimum of 86 pounds and 0.033 inches respectively. All three specimens failed without warning as a longitudinal crack formed between the concrete and steel plate and the load dropped to under 2000 pounds. Shortly after the bond failed, mesh wires began breaking progressively in the typical zipper action which caused complete failure of the ferrocement.

6.3.5 Ferrocement with Shear Studs

Shear transfer from the concrete to steel plate was accomplished in the next two series of tests using shear studs. The average ultimate load of 3830 pounds occurred at a deflection of 0.73 inches (Figure 20). Failure in all three specimens occurred rapidly as the individual wires fractured progressively.

6.3.6 Ferrocement with Epoxy Bond and Cyclic Loading

The repeated load-deflection curves for Specimens 9, loaded so that the steel plate was stressed in compression are shown in Figure 22. The beam was loaded monotonically to its ultimate load of 4100 pounds; this compared well with the ferrocement Specimens 5 shown in Figure 19, which had an average ultimate strength of 4040 psi. Specimens 9b and 9c were loaded cyclically with one cycle of loading to 2000 pounds, one cycle of loading to 3000 pounds, and then loading to ultimate. The load deflection curves in Figure 22 indicate that the intermediate loading and unloading had little effect on the shape of the curve and on the ultimate strength. In all cases failure was rapid with individual wires breaking and producing the zipper action failure mode.

The possibility of concrete deterioration due to a larger number of cycles of loading and unloading prior to ultimate load was considered in Specimens 10 (Figure 23). Specimen 10a was first loaded to 2000 pounds and unloaded five times, then it was loaded to 3000 pounds and unloaded, and finally it was loaded to failure which occurred at a load of 3575 pounds due to bond failure between concrete and steel. Further displacements caused the load to build up again to a maximum of 2580 pounds. Failure here was produced by fracturing of mesh wires which produced the zipper mode of failure. In Specimen 10b bond failure occurred before the 3000 pound load could be achieved. Bond failure occurred in Specimen 10c at 2330 pounds which indicated the possibility of deficient epoxy. Whether the epoxy was sub-standard or not, Specimen 10a showed little concrete deterioration due to repeated load application.

6.3.7 Reinforced Concrete Plus Ferrocement with Epoxy Bond

The load deflection curve for Specimens 11 is shown in Figure 24. A first peak in the curve occurred at an average load of 1800 pounds and a deflection of 0.16 inches. At this point bond failure between the concrete and steel plate occurred with a sudden loss of load. Increased deflections produced increased loads until a second average peak load of 2535 pounds occurred at a deflection of 0.94 inches. At this point mesh wires failed one at a time causing a partial zipper action in the outer layers of mesh.

Additional displacements produced a slight increase in the applied load before complete failure occurred. Thus, the wire mesh controlled crack growth and width until partial zipper action occurred, and then the reinforcing bars controlled the load drop-off at large deflections.

6.3.8 Grade 60 Reinforced Concrete with Epoxy Bond

The deflection curve for Specimens 12 and 13 are shown in Figure 25. Comparing the two curves it is clear that the higher percentage of steel produced little additional strength. Both curves show an average peak load of around 1800 pounds where bond failure between the concrete and steel plate first occurred, reducing the load sharply. Then additional deflections produced increasing loads until average ultimate loads of 2600 pounds and 2750 pounds were reached for the specimens 12 and 13 respectively. At the ultimate load in all four specimens, concrete crushing occurred beneath the steel plate at centerspan.

6.4 Beam With Steel Plate-in-Tension

6.4.1 Reinforced Concrete with Epoxy Bond

The load-deflection curve for Specimens 15, is shown in Figure 26. The average ultimate strength was 4410 pounds. Bond failure between the concrete and steel plate caused the sudden drop in load which occurred at 0.11 inches. After this drop in load, additional deflection caused increased loading to a maximum of 1530 pounds at 1.9 inches deflection.

6.4.2 Ferrocement with Epoxy Bond

The average load-deflection curve for Specimens 14 loaded with the steel plate in tension, is shown in Figure 27. An average ultimate load of 8520 pounds occurred at a deflection of 0.32 inches. At this point bond failure between the concrete and steel plate occurred, reducing the load carrying capacity drastically. Additional deflection produced some increase in load to a maximum of 3400 pounds at a deflection of 1.10 inches.

7. ANALYSIS OF RESULTS

Plain and composite ferrocement and reinforced concrete beams have been analyzed using an ultimate strength procedure, i.e., predictions for ultimate loads and ultimate moments are made from elementary beam theory (with the usual assumptions that the strain in a loaded beam varies linearly across the section and that concrete carries zero tensile load, See Figs. 31 and 32). The analytical predictions are compared with the experimental results to determine the "efficiency of composite action behavior: between, say, steel plate and ferrocement. Knowledge of composite action behavior is necessary for application of the reinforcement ideas to the design of strengthened hulls. Composite action design formulas and their derivation are presented in Chapter 9 of this report.

7.1 Beams With No Steel Plate

7.1.1 Reinforced Concrete

For loads which are applied to the reinforced concrete beams with 2 percent plain steel reinforcement and which produce yielding in the steel, the computed internal beam bending moment is 6.94 inch-kips giving a load at the centerline of 1110 pounds. For external loads which produce ultimate stress in the steel, the computed internal beam bending moment is 9.05 inch-kips giving a computed ultimate load of 1453 pounds. From Figure 17 the average ultimate beam load for reinforced concrete with no plate is 972 pounds which is about 15 percent less than the computed yield load. This deviation may have resulted from slippage of the plain steel bar reinforcement. Longitudinal cracks as well as large flexural cracks developed which indicated that slippage between the concrete and steel reinforcement occurred at steel stresses less than the bar yield stress.

7.1.2 Ferrocement

For purposes of analysis of ferrocement beams, the spacing between layers of mesh was assumed to be equal, and the cover over each outer layer of mesh was assumed equal to 0.05 inches. The fracture strain for the wires was 0.011, and the failure strain for concrete was assumed to be 0.0045. Based on a linear strain distribution, the neutral axis was determined to be 0.276 inches from the compression face of the beam. Mesh strains were computed and converted to stresses using the stress-strain curve in Figure 11. The difference between compression and tension forces in the wire mesh was 15.02 kips. In order for equilibrium to occur, this force along the beam must be carried by concrete in compression. The ultimate strength method with a stress of 0.85 f'_c gave a stress block depth equal to 0.249 inches. The ratio of stress block depth to neutral axis depth measured from the compression face was 0.90. This procedure gave a calculated ultimate bending moment of 11.30 inch-kips, which converted to a center load of 1820 pounds.

The ultimate load capacity of ferrocement beams with no plate was 2290 pounds as shown in Figure 16. This measured ultimate load is about 26 percent larger than the ultimate strength computed above. An explanation of the difference is that the presence of many reinforcement wires controlled crack growth in the cement and concrete matrix. With crack growth controlled it was possible that the concrete was distinctly more effective in carrying tensile stresses and increasing the internal bending capacity of the beam. The increased load carrying capacities cannot be accounted for by the tensile splitting strength of plain concrete. The exhibited synergistic effect of ferrocement is consistent with the observed many fine cracks in the ferrocement beams compared with a few large cracks in reinforced concrete beams.

7.2 Beams With Steel Plate-in-Compression

7.2.1 Reinforced Concrete

For beam loads that produced yield stresses in the reinforcing bars, the neutral axis was 0.166 inches from the compression face, which meant that all of the concrete was in tension. Ignoring tensile strength of concrete and summing internal moments of the beam gave a yield moment of 10.68 inch-kips which corresponded with a center beam load of 1710 pounds. For ultimate stresses in the beam reinforcement, the neutral axis was 0.195 inches from the compression face. The computed ultimate load was 2410 pounds.

The average measured ultimate load of Specimen 6 was 2150 pounds which was between the computed yield and ultimate loads. Again, slip between the reinforcing bars and concrete may have reduced the ultimate capacity In Figure 19 the sudden drop in load at 0.22 inches was due to bond failure at the steel plate concrete interface. The maximum load capacity of the steel plate was 790 pounds (Figure 16) and the average maximum load capacity of plain reinforced concrete was 970 pounds (Figure 17). The capacity of Specimen 6 was 390 pounds (22 percent) over the separate concrete and steel plate strength and was due to composite action of the two beam segments. Friction between the plate and concrete apparently was sufficient to cause the composite action.

A similar method of analysis was applied to Specimens 12 and 13 with three and four #3 reinforcing bars, respectively. The calculated yield load for Specimen 12 was 2750 pounds, compared to the average observed ultimate load of 2600 pounds. The calculated yield load for Specimen 13 was 3660 pounds. compared to the average observed ultimate load of 2750. The difference in both cases was a result of the interface bond failure and subsequent non-composite action. The high percentage of reinforcement in Specimen 13 caused crushing of the concrete to occur before yielding of the reinforcement; this was a more brittle type of failure.

7.2.2 Ferrocement

A one-inch thick slab of ferrocement attached to a 1/4-inch thick steel plate was analyzed by a trial-and-error application of the ultimate strength procedure. A linear strain distribution was assumed for the beam cross-section, and stresses were computed for individual layers of mesh, which were then used to compute the internal bending moments. By such trial-and-error application of linear strain distribution, the neutral axis at ultimate was found to be in the 1/4-inch steel plate about 0.08 inches above the steel-concrete interface. The steel plate was strained beyond yield 0.057 inches from the compression face toward the neutral axis. This portion of the cross-section was assumed to be at the yield stress of 32.9 ksi. The remainder of the plate was considered to be elastic, and the mesh strains were computed using the linear strain variation from the neutral axis. The internal beam bending moment was computed as 23.75 inch-kips which gave a centerline load of 3800 pounds.

The average experimental ultimate load for Specimen 5 was 4040 pounds which compares well with the analytically obtained load of 3800 pounds. The effect of synergism in ferrocement is a possible explanation for the measured strength of ferrocement exceeding the computed strength.

Composite action for the ferrocement beam acting with the steel plate was determined by comparing measured strengths of the combined system with the strengths of the separate components. The load capacity of the steel plate was 790 pounds, and the load capacity of ferrocement with no plate was 2290 pounds. The sum of the strengths of the components is 3085 pounds which is much less than the measured strength of the composite system of 4040 pounds. Thus, the composite action of ferrocement and steel plate was significant and produced an additional 31 percent load carrying capacity.

7.2.3 Ferrocement Plus Reinforced Concrete

Specimens 11 were analyzed by assuming failure in the extreme layer of mesh at a strain of 0.011. A trial-and-error method was used to locate the neutral axis in the steel plate at about 0.10 inch from the concrete/ plate interface. The ultimate moment was calculated to be 22.7 kip-inches which gave an ultimate center load of 3360 pounds. Experimentally, the average ultimate load was 2535 pounds. The difference resulted because interface bcnd failure occurred before the ultimate load was reached. The bond failure reduced the composite action of the beam and, therefore, the load capability.

7.3 Beams with Steel Plate-in-Tension

7.3.1 Reinforced Concrete

Specimens 15, which were constructed with five #2 bars and tested with the plate in tension, were analyzed using an ultimate strength procedure. ultimate strain of 0.0045 was assumed for the concrete. Using a trial- and error method, the neutral axis was located in the concrete section about 0.07 inch from the concrete-plate interface. The ultimate moment was calculated as 56.6 kip-inches, which gave an ultimate center load of 8400 pounds. Specimens 15 actually failed in bond at an average center load of 4410 pounds. Possibly the bond failure at the lower load resulted from poor surface preparation. These specimens were not sandblasted prior to the application of epoxy.

7.3.2 Ferrocement

Again, using an ultimate strain of 0.0045 for cement mortar the neutral axis for Specimens 14 was calculated at about 0.05 inch from the cement-plate interface inside the cement section. The corresponding ultimate moment and center load were 45.9 kip-inches and 7200 lbs., respectively. Experimentally, Specimens 14 exhibited a bond failure at an average center load of 8520. The actual load capability was higher probably because the well-distributed mesh permitted the concrete to strain greater than the assumed 0.0045.

8. DISCUSSION OF RESULTS

8.1 Comparison of Ferrocement and Reinforced Concrete

Four areas and their application to ship hull reinforcement are considered: (1) cracking behavior, (2) failure mode, (3) synergism, and (4) construction.

As discussed in the Background section and as determined by Specimens 2 through 13, ferrocement showed a much greater cracking resistance than reinforced concrete. In ferrocement the flexural cracks were fine and well distributed even at high loads. In reinforced concrete a few large cracks developed in the region of maximum moment. The limitation of crack size may be an advantage in ship hull reinforcement, as corrosive elements including seawater would be prevented from coming into direct contact with the steel reinforcement; this would reduce maintenance and prolong the life of the structure.

The failure mode of ferrocement plate-in-compression composites was brittle; after limited deformation the mesh ruptured, and the beams' load capacities dropped to that of the steel plate alone. Reinforced concrete composites retained their high load capacities at deflections over twice those where the ferrocement failed; reinforced concrete failure was ductile.

This difference between brittle and ductile failure modes is important when considering the working load level and safety factors for the composite sections. Typically high working loads in relation to the ultimate are allowed for structures which fail in ductile modes; low safety factors are selected. Brittle structures are loaded at levels much less than their ultimate; high safety factors are selected. As an example, in Specimens 2 through 8 the high strength ferrocement beams failed at loads nearly twice those of reinforced concrete beams. Yet, a safety factor of 2 might be applied to reinforced concrete, while a factor of 3 to 4 might be applied to ferrocement. The resulting allowable working loads for both types of beams would be about the same. Thus the strength advantage of ferrocement may be negated by its brittle failure mode.

Nevertheless, ferrocement exhibited some improved properties compared to reinforced concrete in both plain and composite beams; these improvements may be considered synergistic effects. One property was that the ultimate compressive strain in ferrocement was greater than that of reinforced concrete as demonstrated in plain beam Specimens 2 and 3, and plate-in-tension Specimens

14 and 15. Apparently the fine wire spacing at the compressive face of the ferrocement beams arrested cracks and prevented the spalling and crushing of the concrete which occurred in the reinforced concrete segments. The ferrocement eventually crushed when the mesh buckled.

A second property was the improved bond between ferrocement and steel plate as compared to reinforced concrete and steel plate in the composite beams. In both plate-in-compression and plate-in-tension beams, the steel-epoxy-concrete bond failed at higher center loads and greater center deflections for ferrocement beams compared to reinforced concrete beams. For each beam the neutral axis and, therefore, the maximum shear stress was calculated to be near the plate-concrete interface. The closely spaced mesh in the proximity of the highly stressed interface region helped assure the compatibility of strain between the plate and concrete. For the plate-in-compression specimens the tensile strain in the concrete near the plate might have eased small cracks with resulting strain irregularities. In reinforced concrete composites, this cracking in the plain concrete near the plate may have initiated the early bond failure. In ferrocement composites, the cracking either was stopped or arrested, thus delaying bond failure.

A real practical difference between reinforced concrete and ferrocement was found when constructing the specimens. The reinforced concrete composite beams were fabricated easily using either low slump (high viscosity) or high slump mortar. Fabricating the ferrocement was difficult. A very high slump mortar was required. In order to obtain full penetration of the mortar through the reinforcing mesh, not more than five layers of mesh could be placed at one time. Then the mortar had to be hand troweled and vibrated into the layers of reinforcement. For ship hull reinforcing the use of ferrocement would require much more labor and more careful control of the mortar than the use of reinforced concrete. Therefore, the overall cost of ferrocement would be greater than reinforced concrete.

Specimens 11 were made using both mesh and bar reinforcement, and the behavior of the beams showed some of the better characteristics of both ferrocement and reinforced concrete. The outer layers of mesh increased the cracking resistance and crack distribution. The mesh failed in a brittle mode, but the beam continued to maintain a high load and behaved in a ductile manner. Thus the bar reinforcement assured the desirable ductile behavior. It is possible that the combined bar-mesh reinforcement scheme may be the best for strengthening ship hulls.

8.2 Composite Action ("Coupling")

Full composite action at ultimate load was achieved in Specimens 7 and 8 with the shear studs and in Specimens 9 with epoxy and sand-blasted plate. Generally the mechanism of shear transfer (bond) failed between the steel plate and concrete sections before the ultimate condition was achieved, i.e., before either the reinforcement failed, the concrete crushed, or the steel plate fully yielded. At low loads the shear transfer was satisfactory. After bond failure for plate-in-compression composites, full composite action was lost, but partial and substantial composite behavior was retained. It is believed that friction between the plate and concrete segment caused the partial shear transfer between the two segments.

Ferrocement Specimens 5 exhibited a strength 31 percent greater than the sum of the individual ferrocement and plate load capabilities. Reinforced concrete Specimens 4 had strengths 22 percent greater than the sum of the individual segments. Although sections of different percent reinforcing steel and relative thicknesses would give different composite strengths, this display of partial composite action was significant.

8.3 Shear Transfer

For plate-in-compression reinforced concrete beams, all types of shear connectors used(epoxy, shear studs and natural bond) produced nearly equal ultimate load results. Ferrocement sections behaved similarly at lower deflections, but at higher deflections the shear studs introduced a little additional flexibility.

8.3.1 Studs

Both reinforced concrete and ferrocement beams with the plate in compression exhibited greatest ductility and energy dissipation capacity when studs were used. Shear studs were considered overall the best method for shear transfer, but are not believed to be the most practical for ship strengthening.

The shear studs required considerable labor to place. Application of studs on a ship's interior would be costly compared to epoxy application, and the increase in load capability by using studs compared to their cost probably would not prove economical.

Construction of ferrocement beams with shear studs was difficult and would also be difficult in ship applications. Although the stud pattern was designed so that the studs would fit neatly into the mesh, placing the mesh over the studs was troublesome.

The number of shear studs used was adequate; the equations given in the background section (Chapter 4) to determine the number of studs were satisfactory.

8.3.2 Natural Bond

Natural bond provided the least shear transfer. The bond failed under low loads; yet, partial composite action was achieved as discussed above, probably because of friction. Use of natural bond (no bond) would be the least costly and might prove the most economical for hull reinforcement; the hull interior only would require cleaning before concrete application.

8.3.3 Epoxy

Sandblasting the steel plate before application of the epoxy appeared to improve the shear transfer. In Specimens 9a, b, and c with sandblasting, the reinforcing mesh fractured before bond failure. In other plate-in-compression ferrocement composites the bond failed before the mesh fractured, although the load difference between bond and wire failure was slight. The plate-in-tension ferrocement composite with sandblasting withstood twice the center deflection and load before interface bond failure as compared to the reinforced concrete composite without sandblasting; in this case the calculated capacity of the reinforced concrete beam was greater than that of the ferrocement beam. While the proximity of the mesh to the interface aided in preventing early bond failure, sandblasting was considered to have improved the bond.

Together with previous research, this study indicates that the steel surface should be sandblasted before application of the epoxy and concrete. In ship strengthening this sandblasting partially accomplishes the cleaning of the hull surface; thorough cleaning is required for epoxy and concrete construction.

8.4 Energy Dissipation

Energy dissipation capacity is an important parameter to consider because it is a measure of the impact resistance and the toughness of a structure. A measure of energy dissipation for ferrocement and reinforced concrete beams attached to steel plates was computed by calculating the area under the load-deflection curves. In order to determine this area at a specified center deflection, a line parallel to the load axis was followed from the curve at the deflection specified, down to zero load and then back to the origin.

Energy dissipation curves for reinforced concrete attached to the steel plate using either shear studs, epoxy at the steel concrete interface, or natural bond (no bond) are plotted in Figure 28. At centerline deflections of 0.2 inches to 0.4 inches the epoxy bonded composite sections dissipated a little more energy. However, at large center displacements, no tangible difference is noted between the natural bond surface and the epoxy bond. Shear studs were flexible enough to increase the energy dissipation at larger deflections, but in no case did the shear stud specimens dissipate more than 20 percent greater energy than the other types.

Energy dissipation curves for ferrocement composite sections are plotted in Figure 29. All three kinds of specimens using the various methods of shear transfer demonstrated nearly the same energy dissipation capabilities. Shear stud specimens dissipated slightly more energy than either the test beams with epoxy bonded interface or the sandblasted steel plate with epoxy bonding. The most significant difference was that epoxy bonded specimens failed at 0.70inch deflection which was a significantly smaller center deflection that the other specimens were able to sustain.

Energy dissipation for reinforced concrete bonded to the steel plate using epoxy is shown in Figure 30, where different curves are for different kinds and percentages of reinforcement. A comparison of the four curves shows that the kind of reinforcement did not affect the energy dissipation substantially and that beams with 2 percent and 3 percent reinforcement develop essentially the same dissipation.

It is apparent that energy dissipation of the composite sections is more a function of the type of reinforcement rather than the kind of shear transfer mechanism from concrete to the steel plate. Both ferrocement and reinforced concrete beams attached to the steel with shear studs demonstrated slightly improved energy dissipation capacities over specimens using the other type shear connectors (Figures 28 and 29). Ferrocement, being stronger than the reinforced concrete, dissipated greater energy at lower deflection levels.

However, the ferrocement reinforcement fractured at low displacements which limited the overall energy dissipation capability. At about one inch deflection both ferrocement and reinforced concrete composite sections tested here dissipated about the same amount of energy.

Before bond failure in the beams, epoxy effectively acted to transfer the shear and to force full composite action. This composite behavior produced stiffer deflection response (Specimens 6 and 8) than found in the beams with only natural bond (Specimens 4). Thus epoxy, even without use of sandblasting, was satisfactory in joining the plate and concrete segments and was better than the natural bond.

9. DESIGN METHOD FOR REINFORCING SHIP HULLS

A method is now presented to determine the required thickness of a ferrocement or a reinforced concrete section for reinforcing metal plates, in particular ship hulls. It is assumed that an adequate bond is provided at the interface.

The total required shell thickness t_r at a specific ice rating for an all steel hull can be determined by the designer from the ice rules of the American Bureau of Shipping (ABS). Using the latter thickness the required bending moment M_R per inch width of plate is given by

$$M_R = f_s t_r^2 / 6 \qquad [9\text{-}1]$$

where f_s is the specified yield strength of steel used in the hull.

If the actual or the existing steel plate thickness t of the ship hull is less than the required thickness t_R obtained above, it will be necessary to provide reinforcement.

Earlier it was pointed out that steel plate reinforced with ferrocement or reinforced concrete has a greater bending moment carrying capacity than the sum of the bending moment capacities of each part taken individually (Table 2). The presence of bond at the interface helps produce this composite action "coupling effect."

Thus, to find the required thickness d of ferrocement or reinforced concrete to withstand M_R, the coupling effect of the composite section, i.e. steel plate-ferrocement or reinforced concrete, must be considered in the analysis and design. Coupling effects due to ferrocement are different from those due to reinforced concrete.

A study of Table 2, shows that composite sections with the steel plate in compression develop smaller ultimate bending moment capacity than with the steel plate in tension. Accordingly, for design purposes only the former need be considered, since it is the critical mode encountered in ice navigation.

9.1 Reinforced Concrete with Steel Plate

The assumed stress distribtion across the critical section of a reinforced concrete with steel plate in compression at yield moment is shown in Figure 31.

The value of x, the distance from the interface to the neutral axis of the steel plate is obtained by equating the sum of the horizontal forces to zero, thus,

$$x = \frac{(t-2pdf_y/f_x)t}{(t-wpdf_y/f_s) + t} \qquad [9\text{-}2a]$$

where, f_y = the specified yield strength of reinforcing bars

d = the distance from the interface of composite section to the centroid of the reinforcing rods

p = the ratio of reinforcing bar area A_s to the area of beam cross section (d times width b), i.e. $p = A_s/bd$

The corresponding yield moment M_Y is obtained by summing the moment of the forces in Figure 31 around the neutral axis, i.e.,

$$M_Y = pdf_y(d+x) + \frac{f_s}{3}\left[(t-x)^2 + \frac{x^3}{(t-x)}\right] \qquad [9\text{-}2b]$$

Calculated load values in Table 2 for Specimens 4,6,8,11,12 and 13 were obtained using Equations [9.2] As can be observed, they compare very well with the experimentally obtained results. Thus, the use of Equation [9-2] is recommended for design of composite sections made of steel plate and reinforced concrete.

9.2 Ferrocement with Steel Plate

It is assumed that the ferrocement is reinforced with many layers of high strength wire mesh uniformly distributed across its section. Once a wire anywhere in the section reaches its ultimate load capacity and breaks no additional load may be supported--unlike the steel plate which is assumed to fail only after full yield has developed throughout

In this analysis, since the wire mesh is considered uniformly spread, it will be "smeared" across the section giving a new equivalent ultimate stress

$$S_u = pf_u$$

where, p = the ratio of reinforcing mesh wire cross sectional area A_s, to gross ferrocement area bd, i.e., $p = A_s/bd$

and f_u = wire ultimate strength.

The stress distribution corresponding to the above assumptions across the critical section of a ferrocement with steel plate in compression at ultimate moment is shown in Figure 32.

As before, the value of x is obtained by equating the sum of the horizontal forces equal to zero, and is given by

$$x = \frac{-n-(n^2-mq)^{0.5}}{2m} \qquad [9\text{-}3a]$$

where $m = 4C-2t$

$n = t^2+2Cd=2td-4Ct$

$q = t^2d-2Ctd$

and $C = \frac{pf_ud}{2f_s}$

The ultimate moment M_U is obtained by summing the moments of the distributed stresses shown in Figure 32 around the neutral axis and is expressed by

$$M_U = pf_ud\,\frac{(d^2+3dx+3x^2)}{3(d+x)} + f_s\,\frac{[x^3+(t-x)^3]}{3(t-x)} \qquad [9\text{-}3b]$$

Values of the ultimate load were calculated for Specimens 5,7,9 and 10 using Equations [9-3]. They are shown in Table 2 and agree quite well with the experimental results. Its use for design of composite sections made of steel plate and ferrocement is likewise recommended.

For proportions of steel plate thickness to concrete depth and material properties much different than used in these tests, the validity of Equations [2] and [3] must be verified experimentally by further tests.

9.3 Design examples

In this section Equations [9-2] and [9-3] are applied to a practical design problem.

Consider a ship having a hull plate thickness of t = 0.530 inch. For a transverse frame spacing of 22 inches, ice pressure of 234.5 psi and steel plate yield stress of f_s = 36,000 psi, according to American Bureau of Shipping (ABS) ice rules, the required thickness t_r is 1.284 inches

Since t_r is greater than t the hull plate needs reinforcing. The required bending moment is obtained from Equation [9-1] as

$$M_R = \frac{1.284^2}{6} \cdot 36,000 = 9892 \text{ in-lb.}$$

9.3.1 Using Reinforced Concrete

Let the yield strength f_y of reinforcing rods to be used, equal 60,000 psi with a value of $p = 0.02$.

We now equate the value of M_R above to M_Y in equation [9-2b] and together with Equation [9-2a] we solve for the required depth of reinforced concrete cover and obtain the value of $d = 2.50$ inches.

9.3.2 Using Ferrocement

Assume the ultimate wire strength f_u of the wire mesh to be used is 180,000 psi with $p = 0.02$ as above.

To obtain the ferrocement depth necessary to reinforce the existing plate we equate the value of M_R above to M_U in Equation [9-3b] and with the help of Equation [9-3a] obtain $d = 2.44$ inches.

Note that, for the same percent reinforcing and physical dimensions the steel plate with ferrocement exhibited slightly greater load carrying capacity than with reinforced concrete. The latter behavior is attributed to the high strength quality of the wire mesh used in the ferrocement.

10. CONCLUSIONS AND RECOMMENDATIONS

As a result of testing thirty-four reinforced concrete and ferrocement composite beams, the following general conclusions may be made:

1) With proper bond at the interface between steel plate and concrete the composite section acts monolithically. It develops full composite or "coupling" action until the ultimate load is reached.

2) Method of shear transfer (bond mechanism) from concrete to steel plate does not substantially alter the ultimate load carrying ability of the composite section. Sandblasted steel plate plus epoxy cement was considered adequate in achieving full composite coupling action.

3) Cyclic loading, tested to 70% of the ultimate load, did not cause significant reduction in the ultimate load carrying capacity of the composite sections.

4) Energy dissipation depends more on the type of steel reinforcement used than the amount of steel (rod or wire) reinforcement or method of shear transfer at the interface of steel and concrete employed.

5) Simplified analytical models (Equations 9-2 and 9-3), are capable of adequately predicting the ultimate behavior of such composite sections.

Thus to design reinforcement for a ship hull against ice loading, Equations 9.2 (for concrete) or Equations 9-3 (for ferrocement) may be used as alternatives to reinforcement with extra plain steel.

If the anticipated deflection is not very large and hull stiffness is the main concern, then ferrocement would be more desirable in that it is much stronger than reinforced concrete for the same depth of concrete and percentage steel (rod or wire) cross sectional area. On the other hand, reinforced concrete composite sections have greater ability to sustain loads at large deflections.

Cost of application also is a factor, much more labor being involved in constructing ferrocement than reinforced concrete. Again, in order to increase the toughness of either ferrocement or reinforced concrete composite sections, the use of shear studs would be desirable, which are costly.

It is recommended to study the effect of varying the depth of concrete used as well as its ultimate strength. These were kept constant in the experiments reported here. The strength parameter of concrete may be significant in the ultimate load carrying capacity of such composite sections. However, because the neutral axes of the specimens used were located in the steel section, Equations 9-2 and 9-3 were unable to account for it.

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LIST OF TABLES

Table 1: Material Properties*

Specimen	Concrete Strength (psi)			Reinforcement Strength (psi)	
	7-day	28-day f'_c	Tensile Splitting	Yield**	Ultimate
1	--	--	--	--	--
2	4,260	6,220	--	144,000	184,200
3	3,950	6,320	--	51,100	73,000
4	3,950	6,320	--	51,100	73,000
5	--	6,100	510	144,000	184,200
6	4,320	6,270	--	51,100	73,000
7	--	4,150	340	144,000	184,200
8	3,720	5,320	--	51,100	73,000
9	--	5,560	460	144,000	184,200
10	3,760	5,000	--	144,000	184,200
11	4,640	6,230	--	144,000	184,200
12	--	6,160	480	69,600	101,200
13	--	6,160	480	69,600	101,200
14	--	5,660	545	144,000	184,200
15	4,320	6,270	--	51,100	73,000

*The yield strength of the steel plate was 32,900 psi for each specimen.

**At 0.2% offset

TABLE 2 : Experimental Program

Specimen	Concrete Reinforcement	Percent Reinf.	Steel Plate Stress	Shear Connection	Loading	Ultimate Load (lbs)			Calculated Load (lbs.)
						Average	Maximum	Minimum	
1a,b	--	--	Flexure	--	Monotonic	790	790	790	610
2a,b,c	10 L[1] mesh	1.96	--	--	Monotonic	2290	2400	2200	1820
3a,b,c	5 #2 bars	2.05	--	--	Monotonic	970	1210	740	1110
4a,b,c	5 #2 bars	2.05	Comp.	Natural	Monotonic	2080	2150	2040	1930*
5a,b,c	10 L mesh	1.96	Comp.	Epoxy	Monotonic	4040	4090	4000	3130*
6a,b,c	5 #2 bars	2.05	Comp.	Epoxy	Monotonic	2150	2440	1970	1930*
7a,b,c	10 L mesh	1.96	Comp.	Studs	Monotonic	3830	3960	3710	3130*
8a,b,c	5 #2 bars	2.05	Comp.	Studs	Monotonic	2460	2550	2360	1930*
9a,b,c	10 L mesh	1.96	Comp.	Epoxy[2]	Cyclic	3930	4100	3730	3130*
10a,b,c	10 L mesh	1.96	Comp.	Epoxy	Cyclic	2830	3575	2330	3130*
11a,b,c	5 L mesh 5 #2 bars	3.03	Comp.	Epoxy	Monotonic	2540	2580	2500	3185*
12a,b	3 #3 bars	2.75	Comp.	Epoxy	Monotonic	2600	2750	2450	2275*
13a,b	4 #3 bars	3.67	Comp.	Epoxy	Monotonic	2750	2910	2600	2980*
14a,b,c	10 L mesh	1.96	Tension	Epoxy[2]	Monotonic	8529	9430	7960	7220
15a,b,c	5 #2 bars	2.05	Tension	Epoxy	Monotonic	4410	5920	2880	8400

1. L: layer of 4 X 4 X 0.025 wire mesh
2. plate was sandblasted
3. Specimen 9a loaded monotonically

*Yield load for reinforced concrete and ultimate load for ferrocement from Equations 9-2 and 9-3.

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Fig. 18 Average load-deflection response of reinforced concrete, plate-in-compression Specimens 4 made using natural bond

Fig. 19 Average load-deflection response of ferrocement, plate-in-compression Specimens 5, and reinforced concrete, plate-in-compression Specimens 6, both made using epoxy

Fig. 20 Average load-deflection response of reinforced concrete, plate-in-compression Specimens 8, made using 1/8 inch shear studs

Fig. 21 Average load-deflection response of ferrocement, plate-in-compression Specimens 7 made using 3/16 inch shear studs

Fig. 22 Load-deflection response of ferrocement, plate-in-compression Specimen 9a loaded monotonically and 9b loaded cyclically. Specimens 9 were made using epoxy and a sandblasted plate.

Fig. 23 Load-deflection response of ferrocement, plate-in-compression Specimen 10a made using epoxy and loaded cyclically

Fig. 24 Average load-deflection response of plate-in-compression. Specimens 11 made with 5 layers of mesh and 5 #2 bars and using epoxy

Fig. 25 Average load-deflection response of reinforced concrete plate-in-compression. Specimens 12 and 13 made with 3 #3 bars and 4 #3 bars, respectively, and both using epoxy.

Fig. 26 Average load-deflection response of reinforced concrete, plate-in-tension. Specimens 15 made using epxoxy.

Fig. 27 Average load-deflection response of ferrocement, plate-in-tension. Specimens 14 made using epoxy and sandblasted plate.

Fig. 28 Energy dissipation of reinforced concrete, plate-in-compression composites using various shear connectors.

Fig. 29 Energy dissipation of ferrocement, plate-in-compression composite using epoxy, plus sandblasted plate, and stud shear connections.

Fig. 30 Energy dissipation of reinforced concrete, plate-in-compression composites using epoxy; made with various amounts of reinforcement

Fig. 31 Stress distribution at yield moment for reinforced concrete with steel plate in compression

Fig. 32 Stress distribution at ultimate moment for ferrocement with steel plate-in-compression

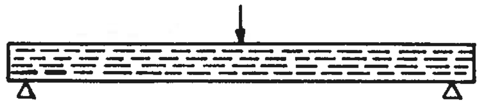
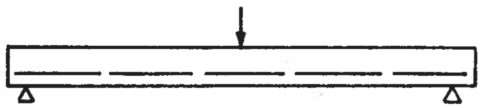
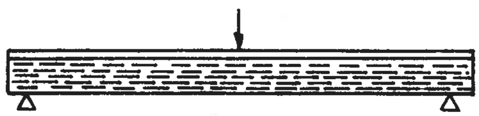
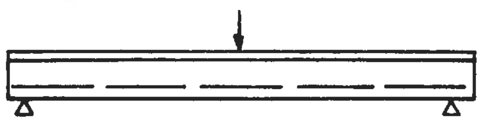
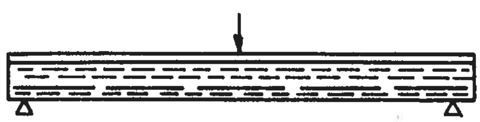
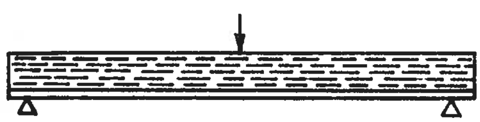
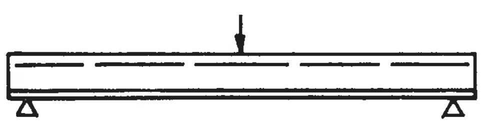
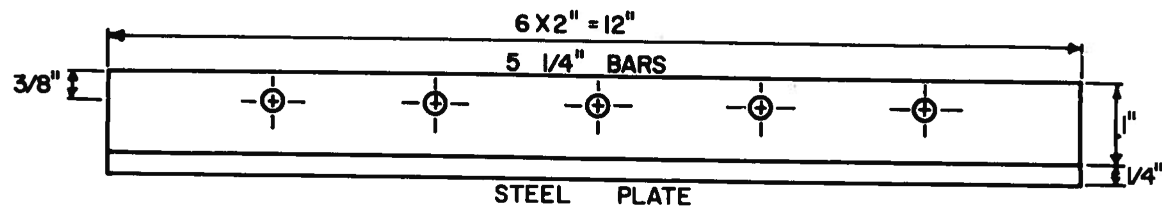
SPECIMEN NUMBER AND SHEAR CONNECTION		EXPERIMENT	SPECIMEN TYPE
Spec. 1			Steel Plate
Spec. 2			Ferrocement
Spec. 3			Reinforced Concrete
Spec. 5,9,10 Spec. 7	Epoxy Studs		Plate-in-Compression Ferrocement
Spec. 4 Spec. 6,12,13 Spec. 8	Natural Epoxy Studs		Plate-in-Compression Reinforced Concrete
Spec. 11	Epoxy		Plate-in-Compression Ferrocement and Reinforced Concrete
Spec. 14	Epoxy		Plate-in-Tension Ferrocement
Spec. 15	Epoxy		Plate-in-Tension Reinforced Concrete

Fig. 1 Experimental Program



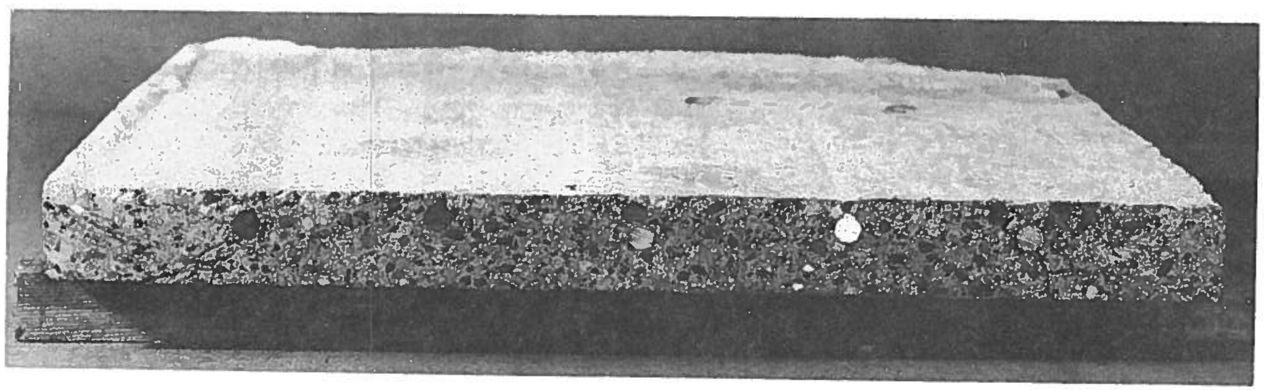
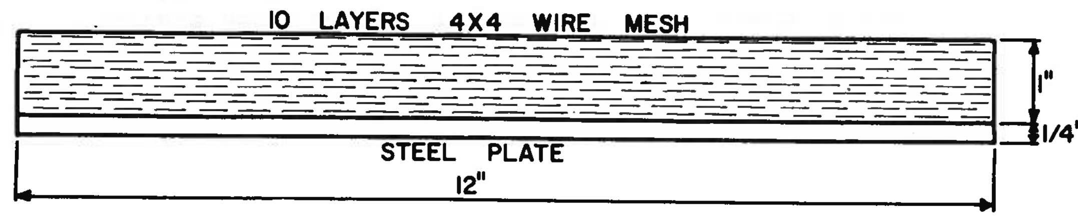


Fig. 2 Reinforced concrete composite section with 5 #2 plain bars (Specimens 4, 6, 8 and 15)



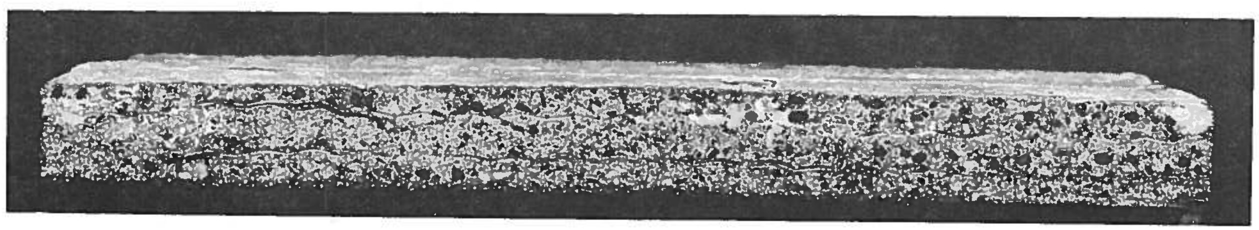
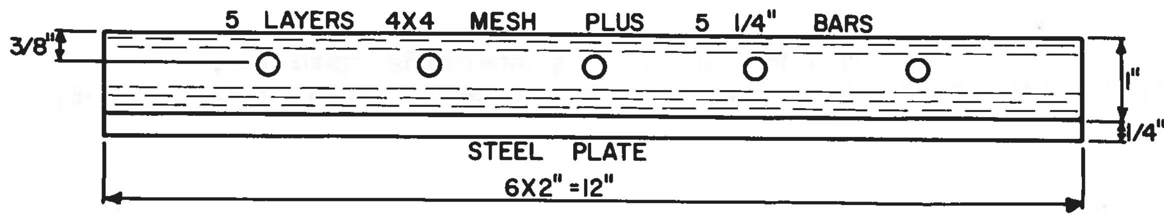


Fig. 3 Ferrocement composite section with 10 layers of 4 X 4 X 0.025 inch mesh (Specimens 5, 7, 9, 10 and 14)



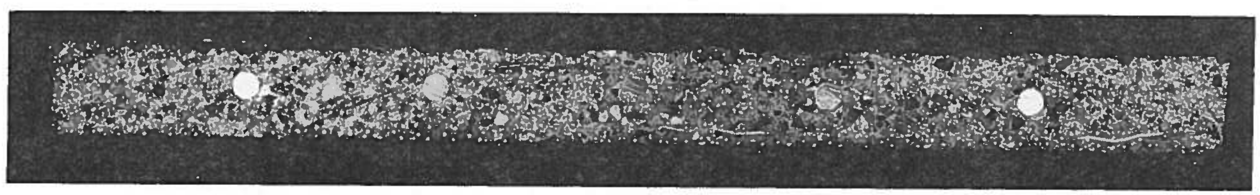
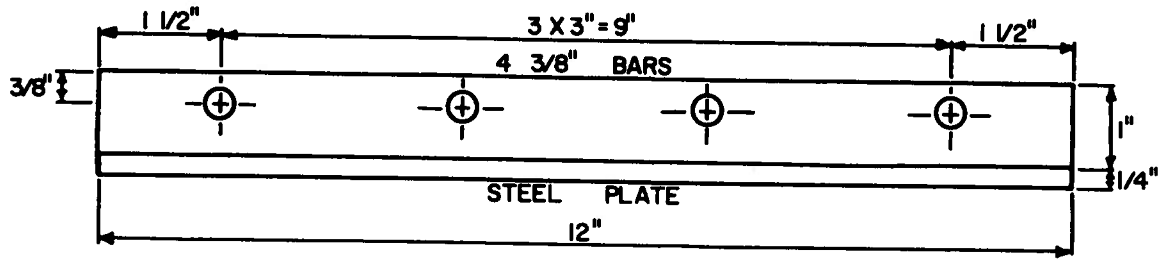


Fig. 4 Concrete section with 5 layers of mesh plus 5 #2 bars (Specimens 11)



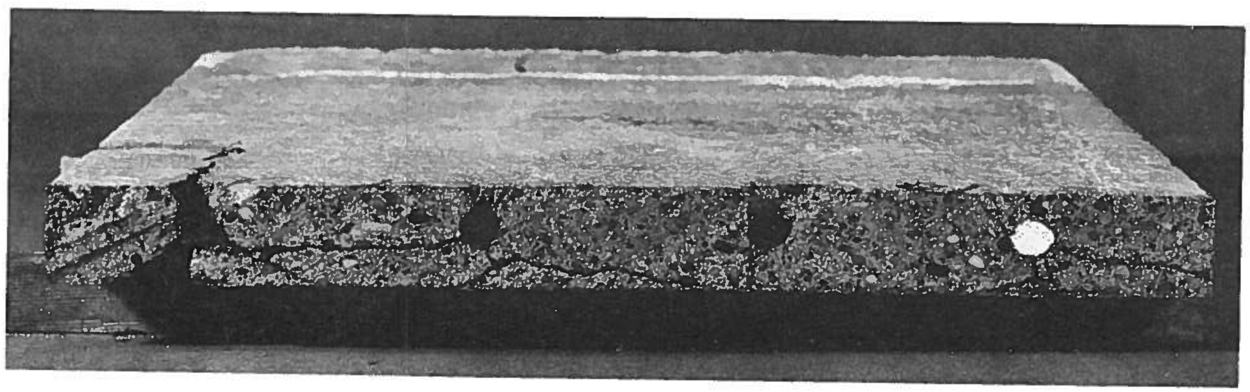
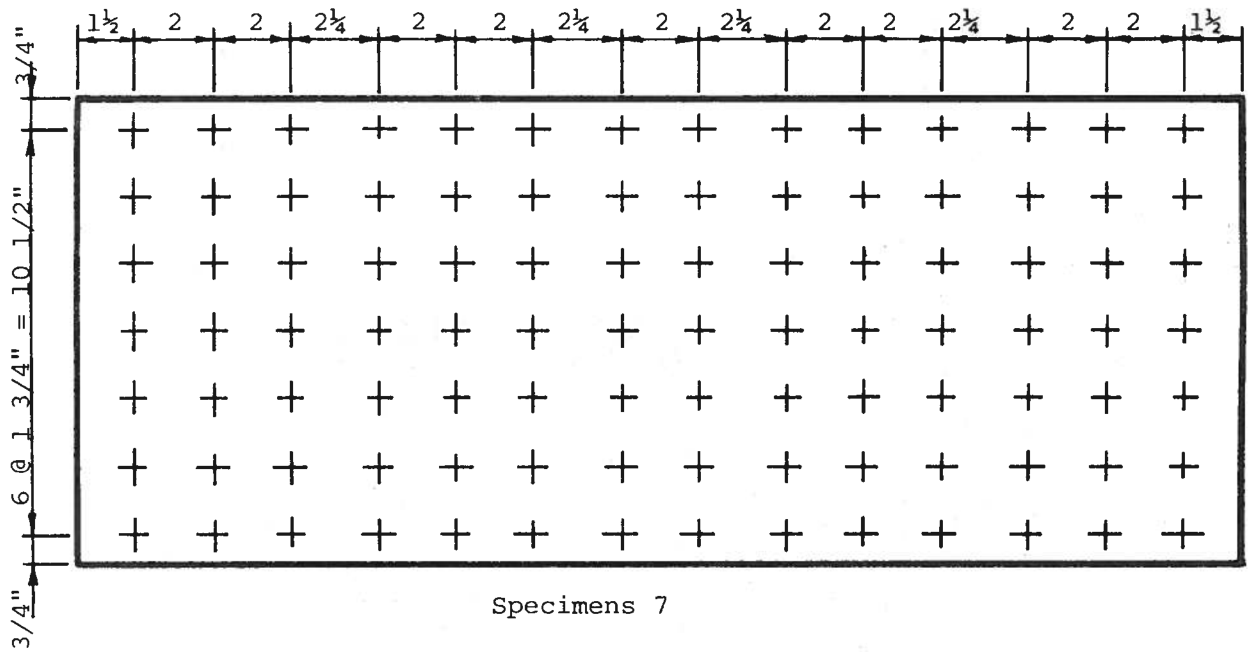
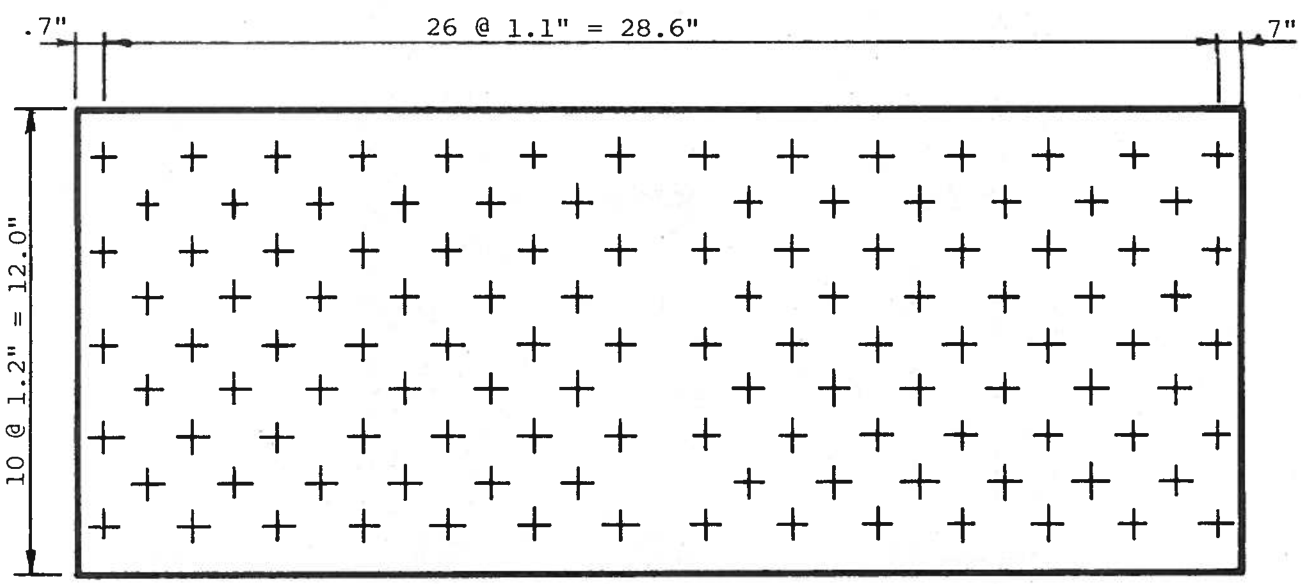


Fig. 5 Reinforced concrete section with 4 #3 deformed bars (Specimens 13)



Specimens 7



Specimens 8

Fig. 6 Shear stud pattern for Specimens 7 (top) and Specimens 8 (bottom)

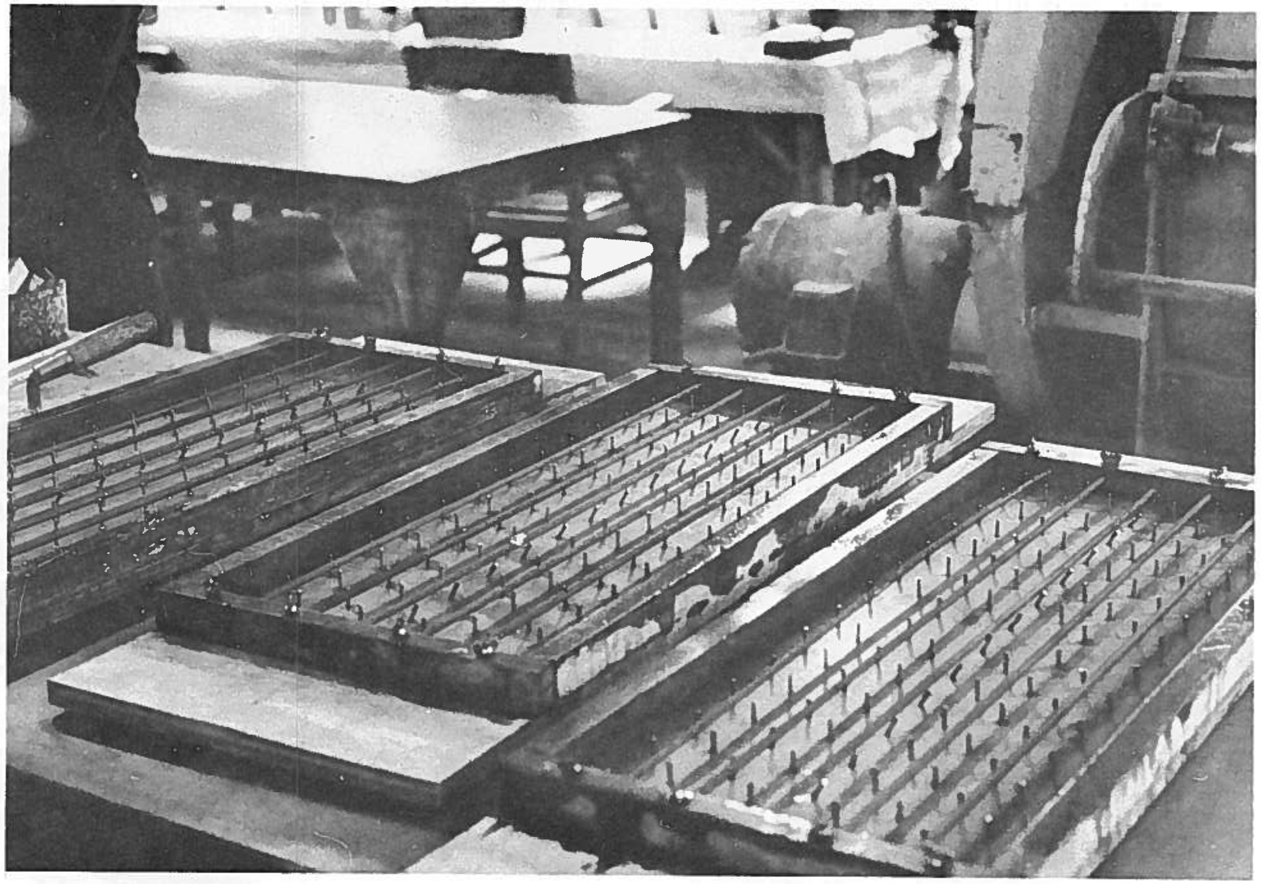


Fig. 7 Photograph of Specimens 8 prior to casting which shows 1/8 inch studs hammered into the plate

Fig. 8 Photograph of Specimens 5 during casting. Mortar was vibrated into the mesh by holding a spud vibrator on top of the mesh

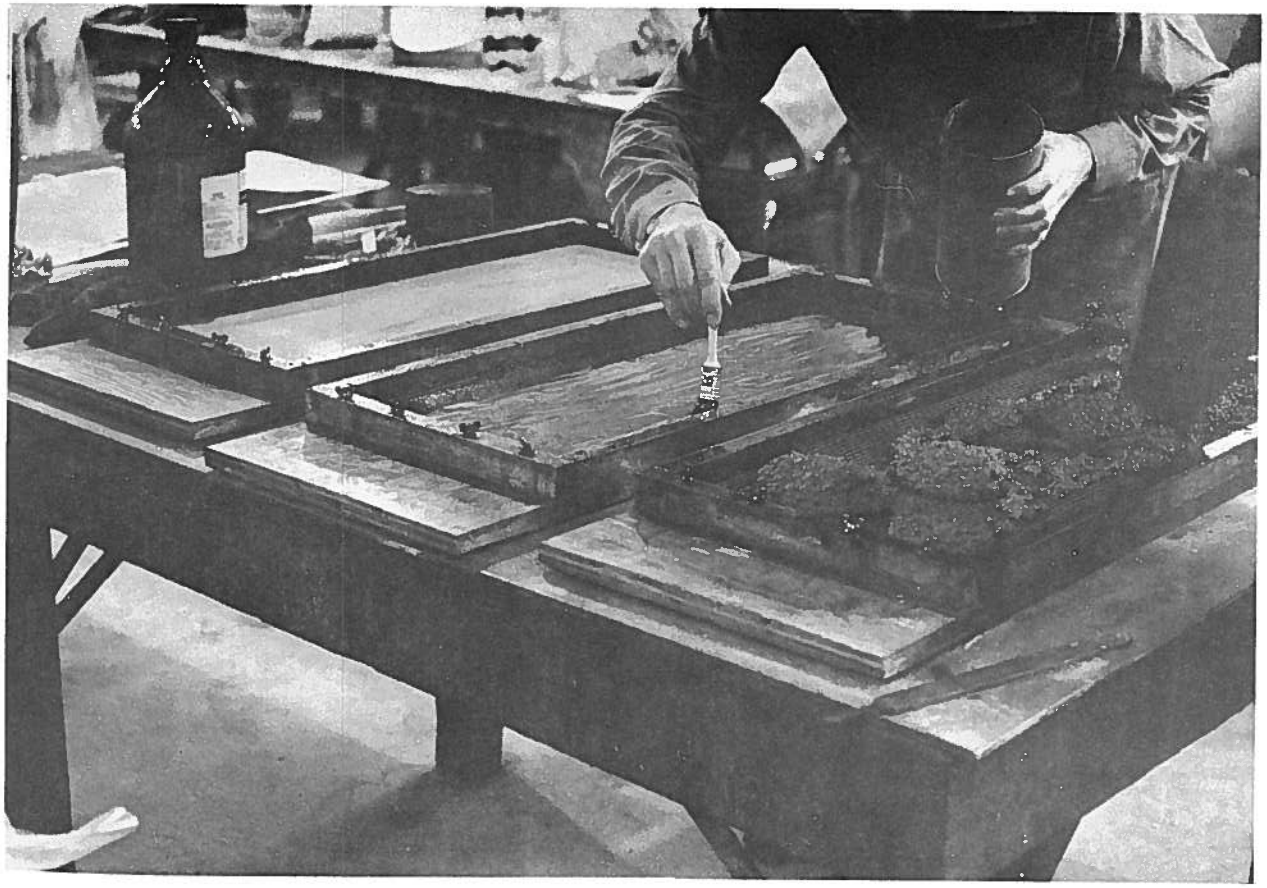


Fig. 9 Photograph of Specimens 5 during casting. Epoxy was painted on the steel plate before placement of mesh and concrete

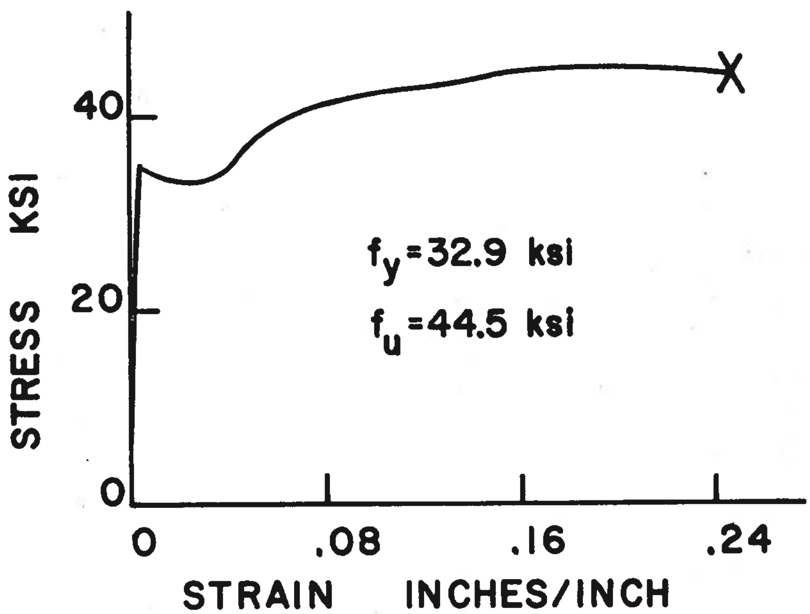


Fig. 10 Stress-strain behavior of the plate steel with yield stress of 32,900 psi

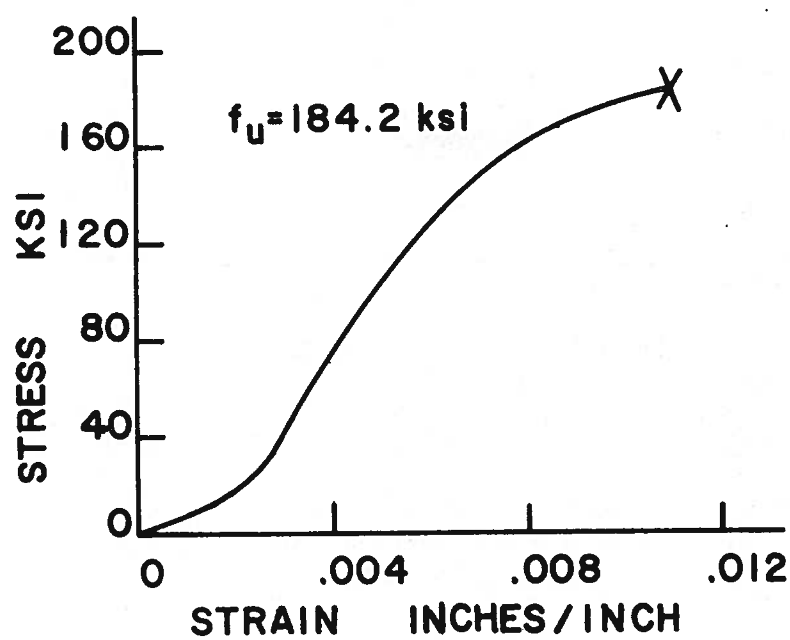


Fig. 11 Stress-strain behavior of the 0.025 inch diameter mesh wire with ultimate stress of 184,200 psi

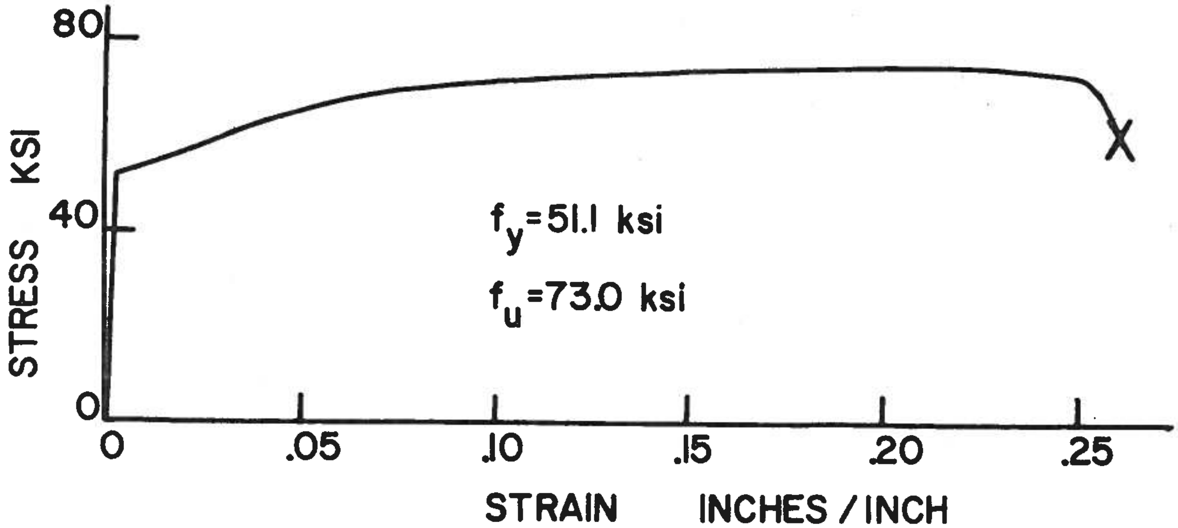


Fig. 12 Stress-strain behavior of plain #2 steel bar reinforcement with yield stress of 51,100 psi

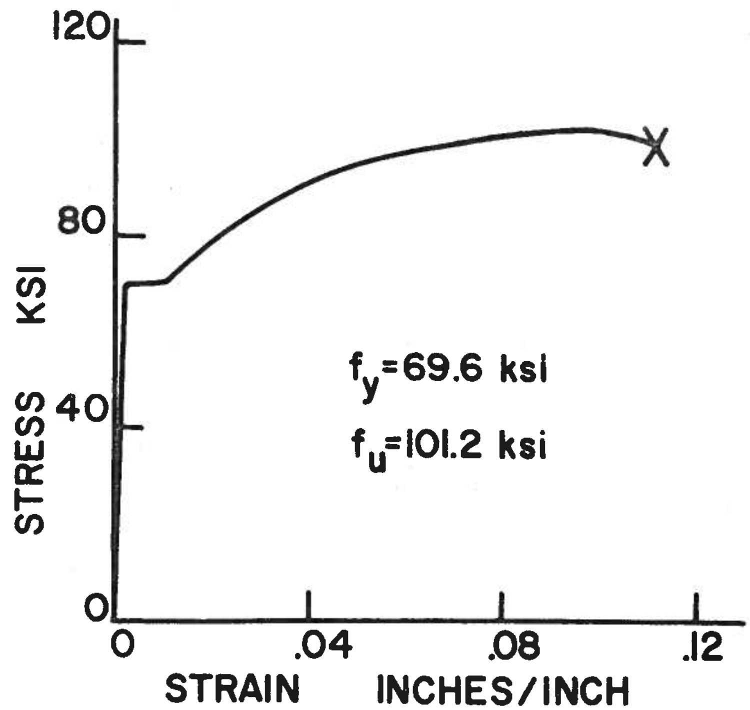


Fig. 13 Stress-strain behavior of deformed #3 steel bar reinforcement with yield stress of 69,600 psi

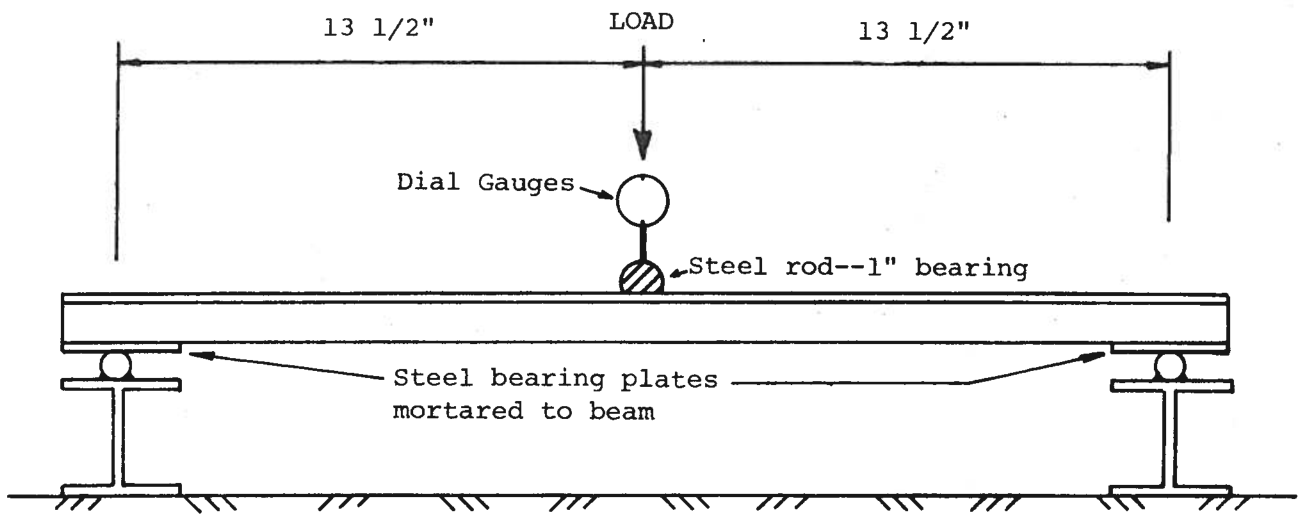


Plate-in-Compression

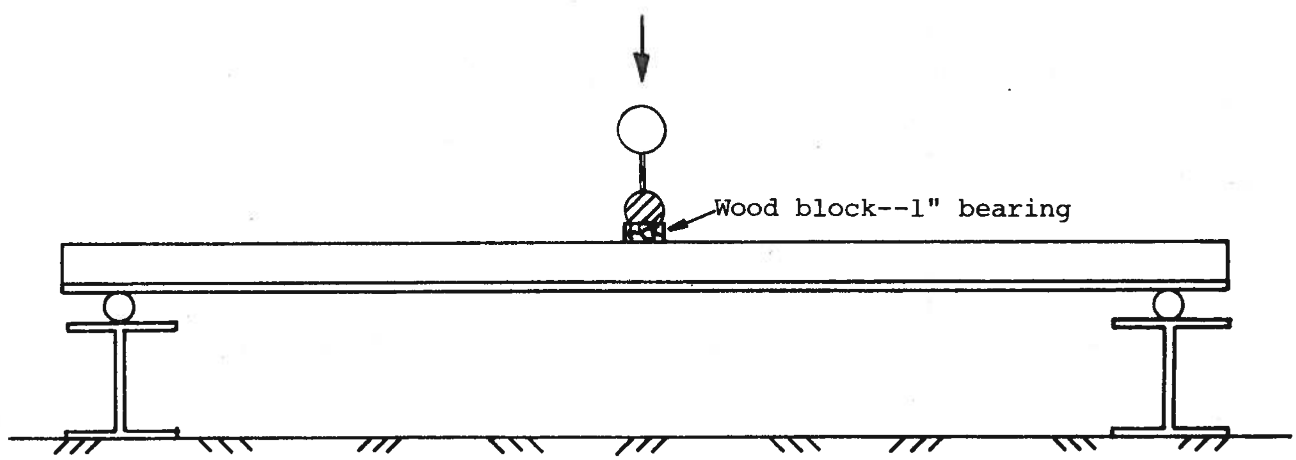


Plate-in-Tension

Fig. 14 Diagram of flexure test arrangements for plate-in-compression (top) and plate-in-tension (bottom)

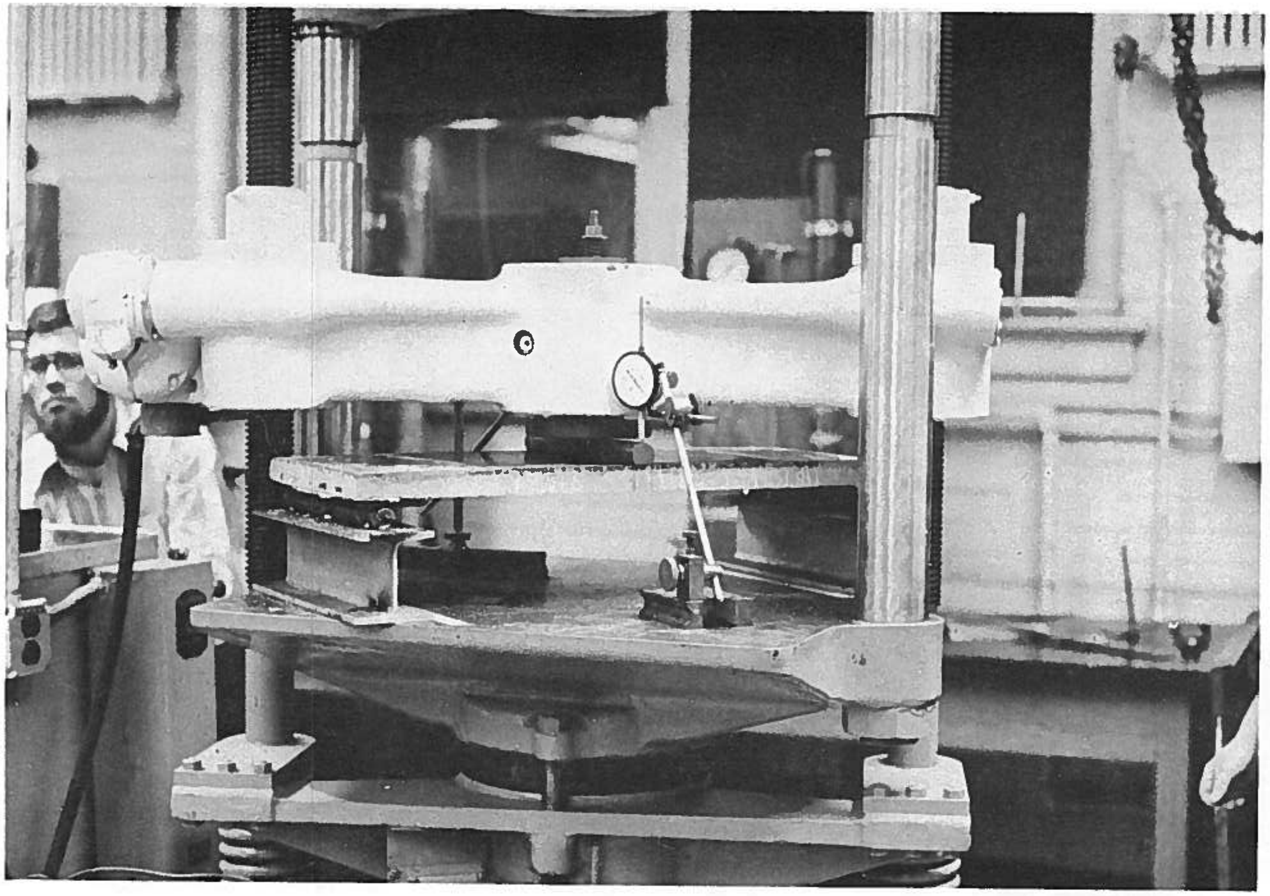


Fig. 15a Specimen 6a being tested with a universal testing machine. Center deflections determined with two dial gauges.

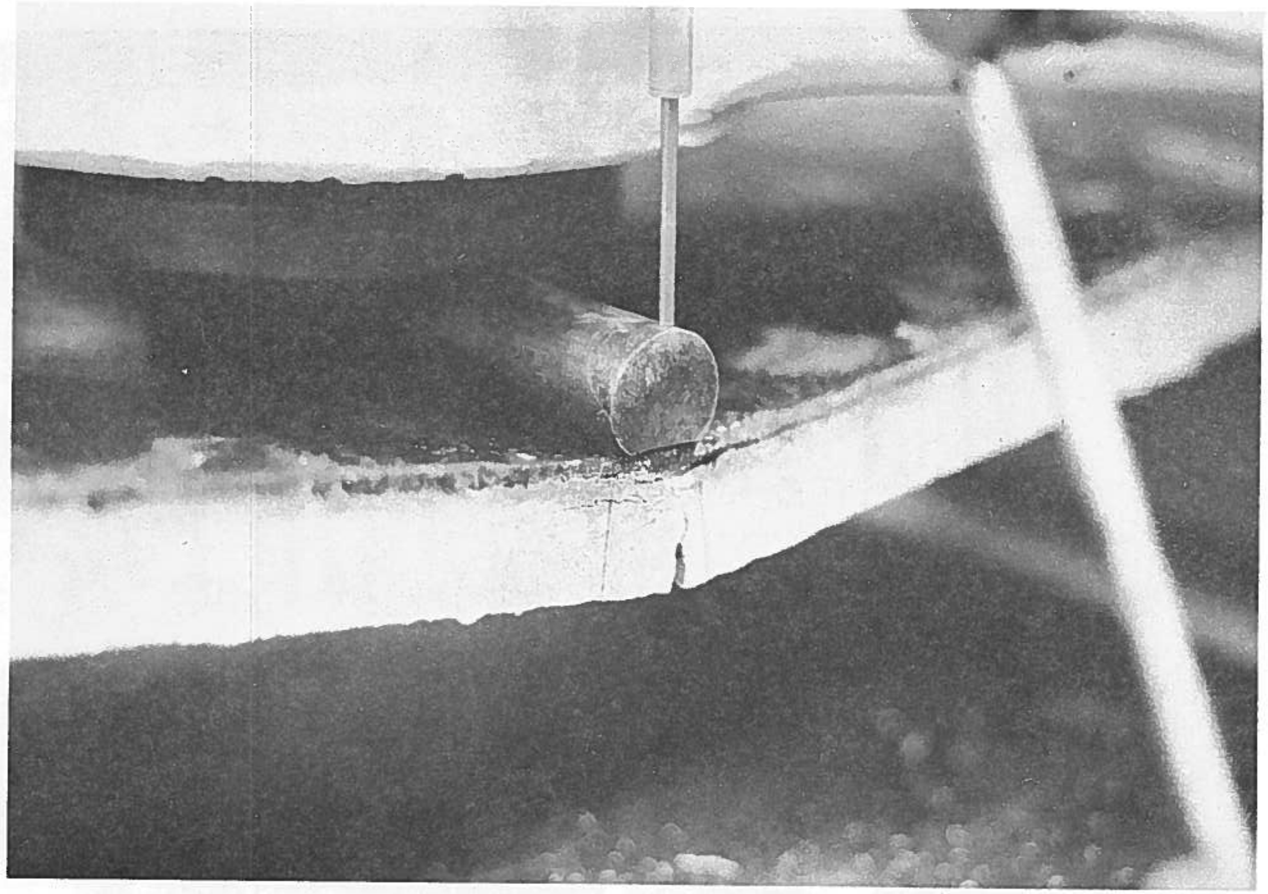


Fig. 15b   Specimen 6a above after failure

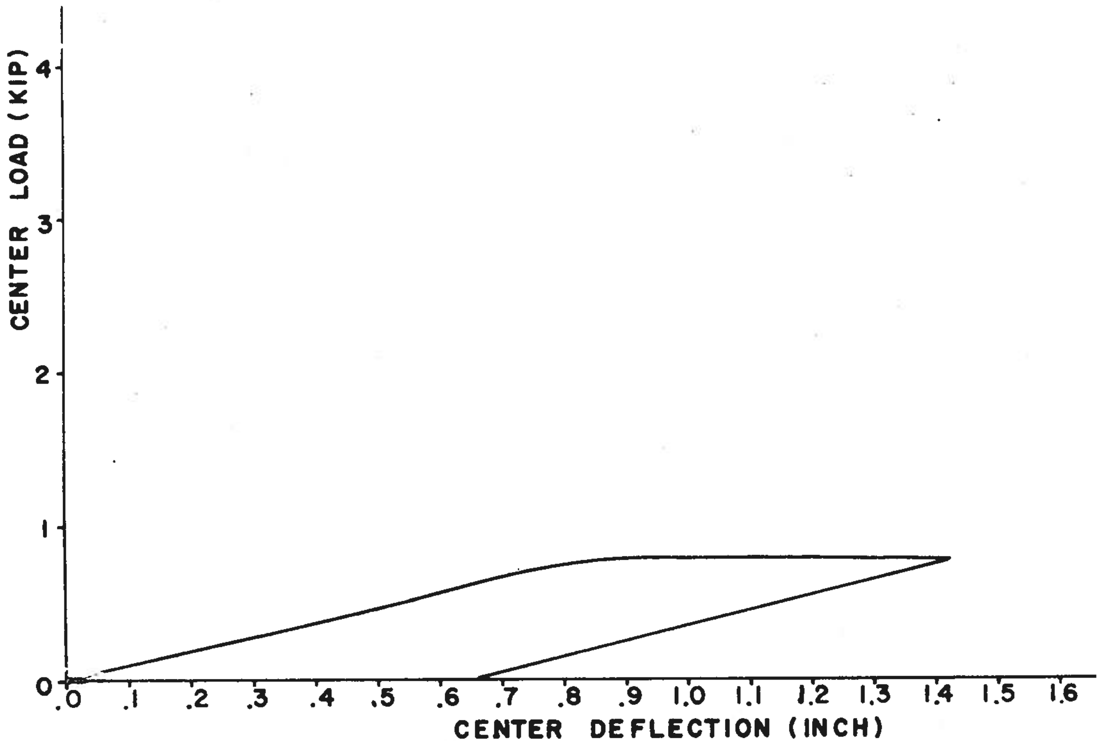


Fig. 16 Average load-deflection response of 1/4 inch steel plate without concrete section (Specimens 1)

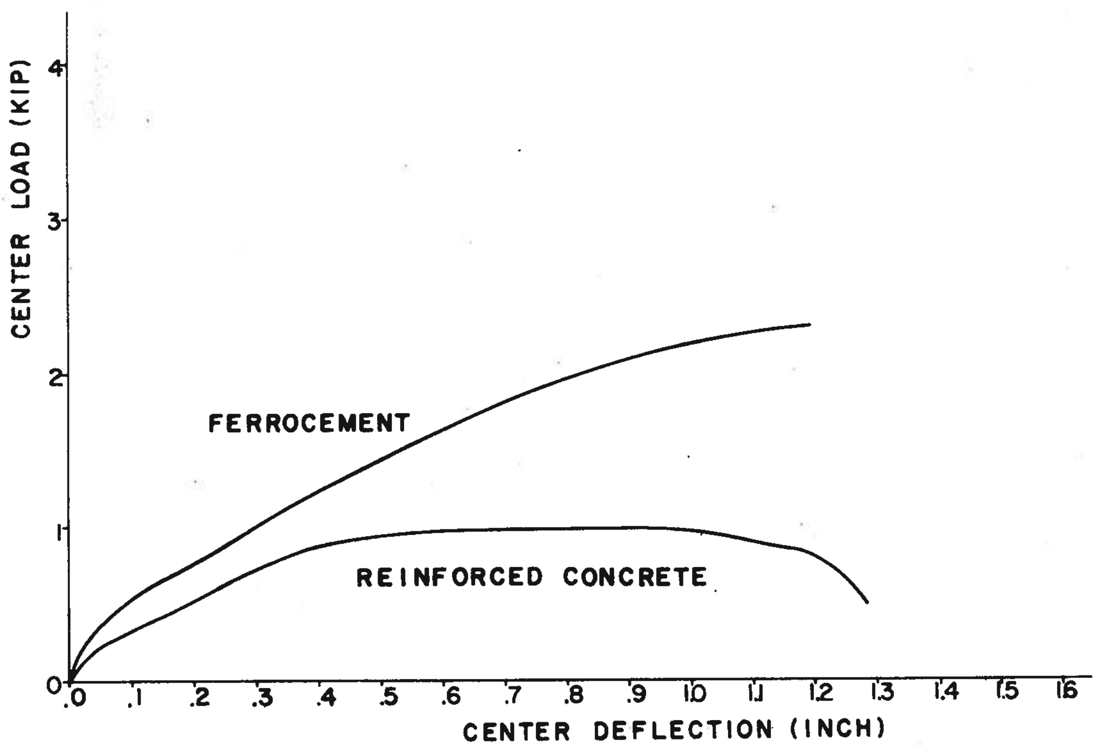


Fig. 17 Average load-deflection response of plain ferrocement (Specimens 2) and reinforced concrete (Specimens 3)

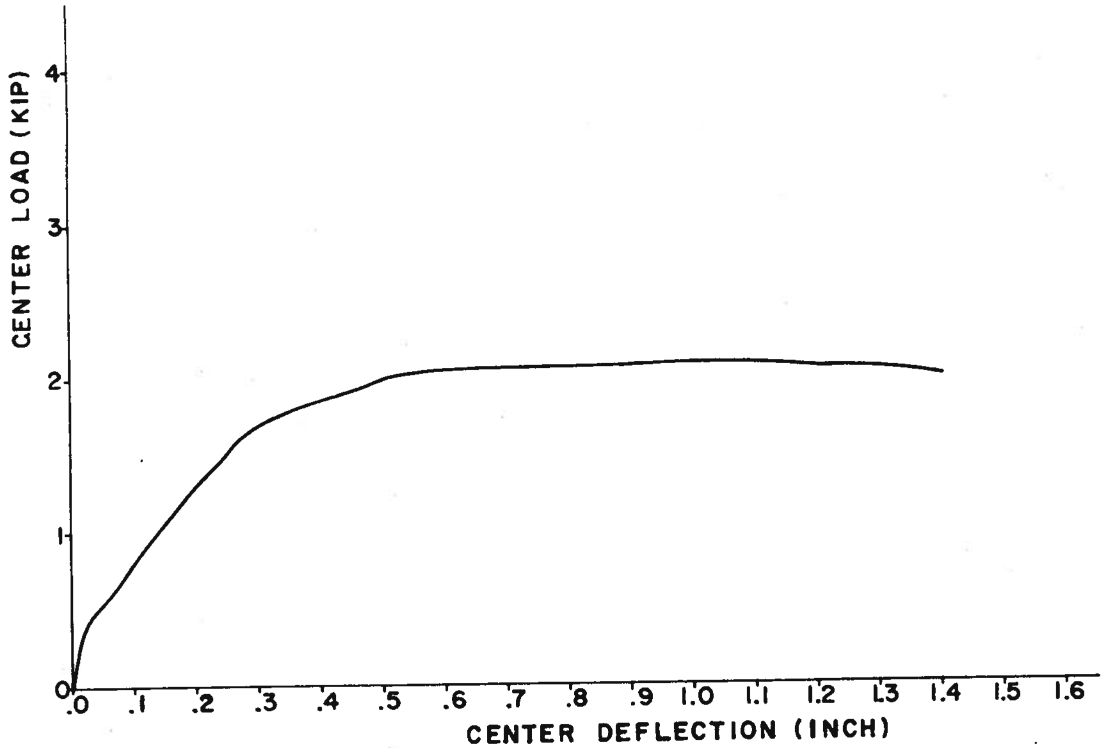


Fig. 18 Average load-deflection response of reinforced concrete, plate-in-compression Specimens 4 made using natural bond

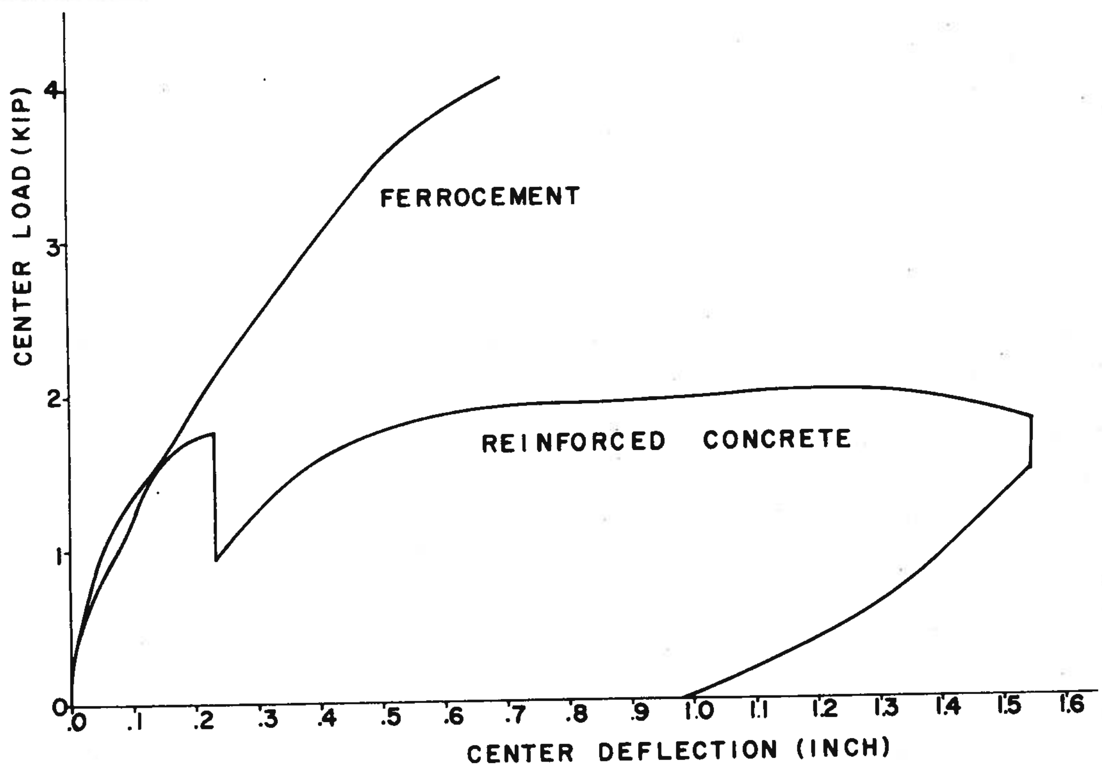


Fig. 19 Average load-deflection response of ferrocement, plate-in-compression Specimens 5, and reinforced concrete, plate-in-compression Specimens 6, both made using epoxy

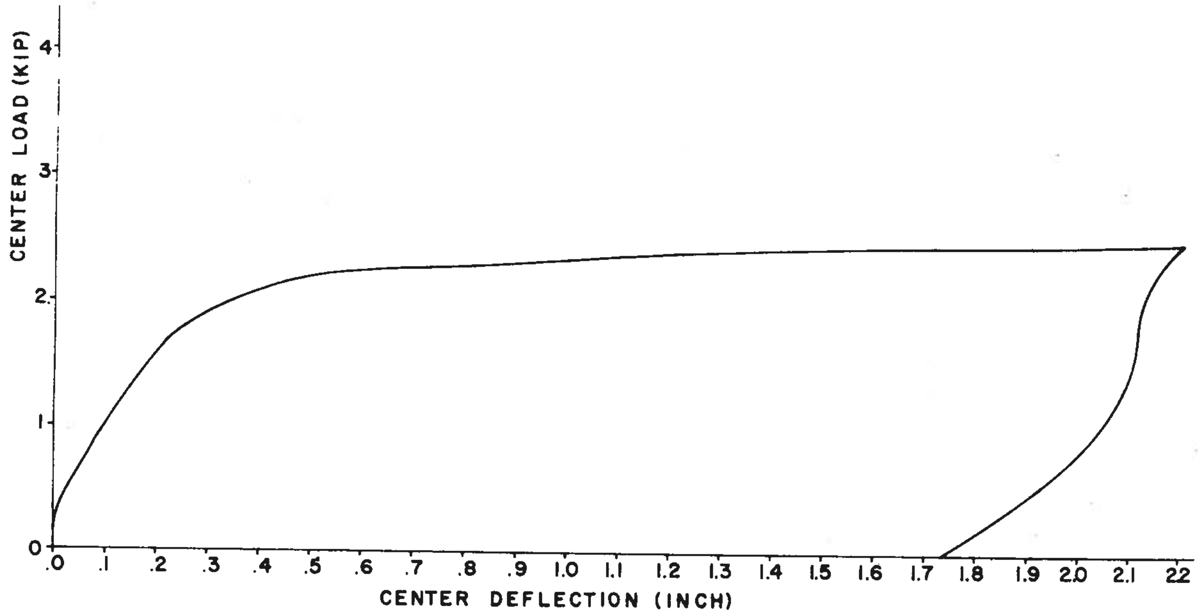


Fig. 20 Average load-deflection response of reinforced concrete, plate-in-compression Specimens 8, made using 1/8 inch shear studs

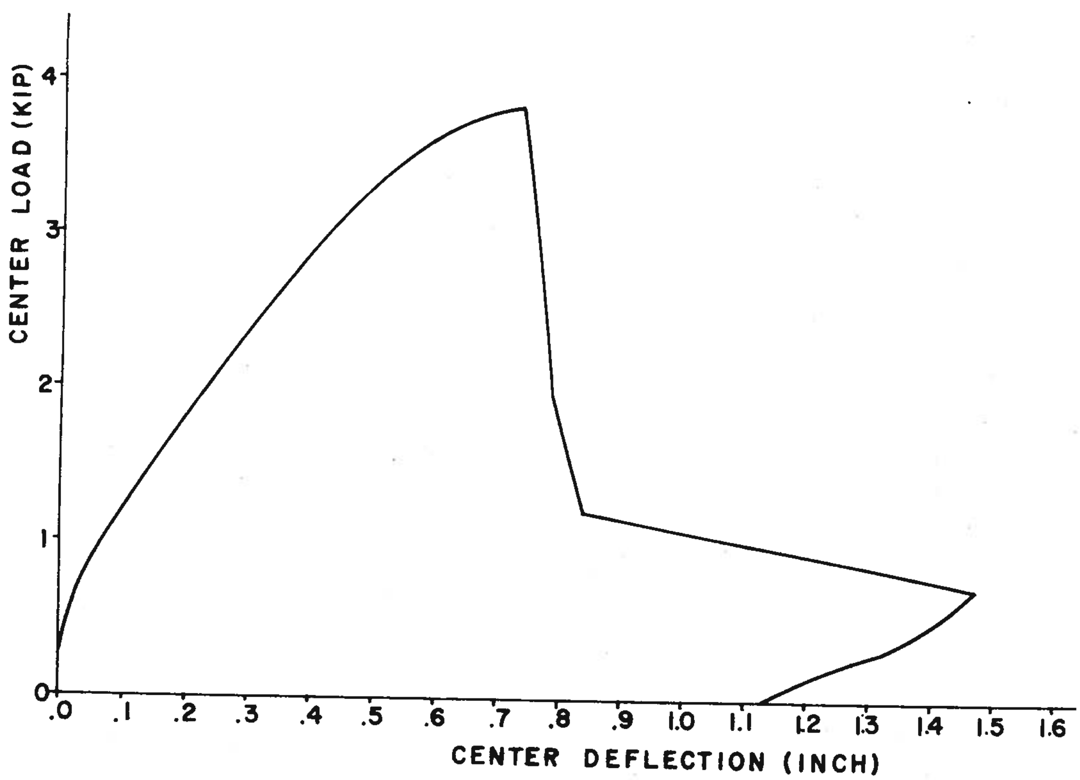


Fig. 21 Average load-deflection response of ferrocement, plate-in-compression Specimens 7 made using 3/16 inch shear studs

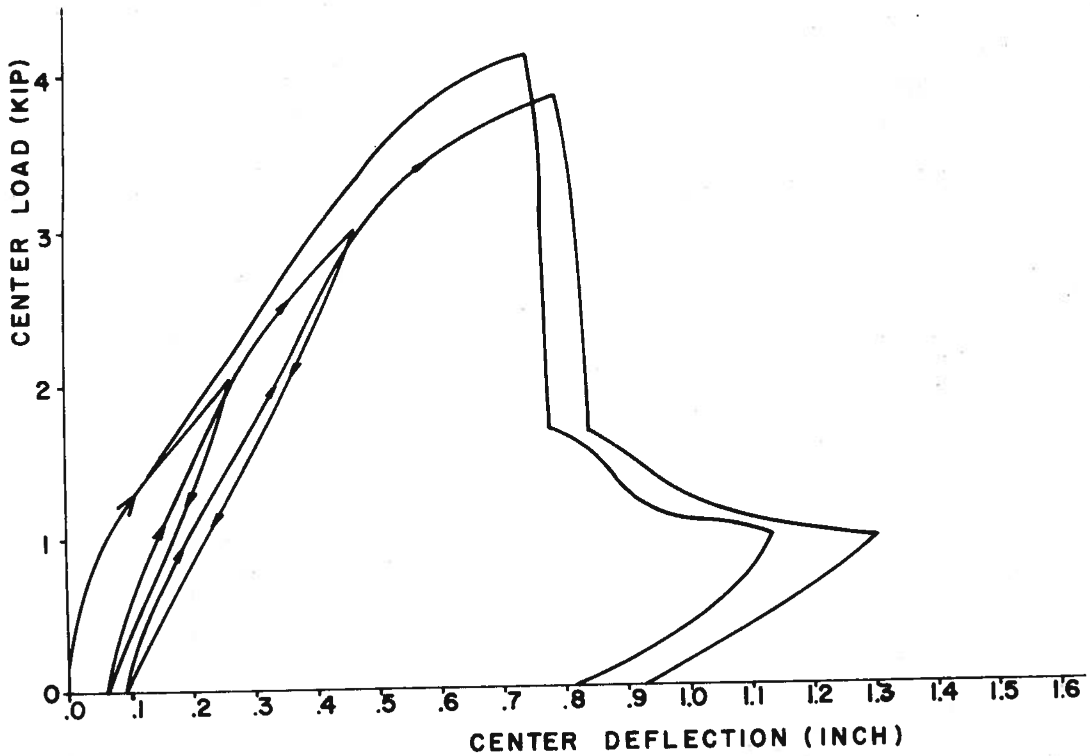


Fig. 22 Load-deflection response of ferrocement, plate-in-compression Specimen 9a loaded monotonically and 9b loaded cyclically. Specimens 9 were made using epoxy and a sandblasted plate

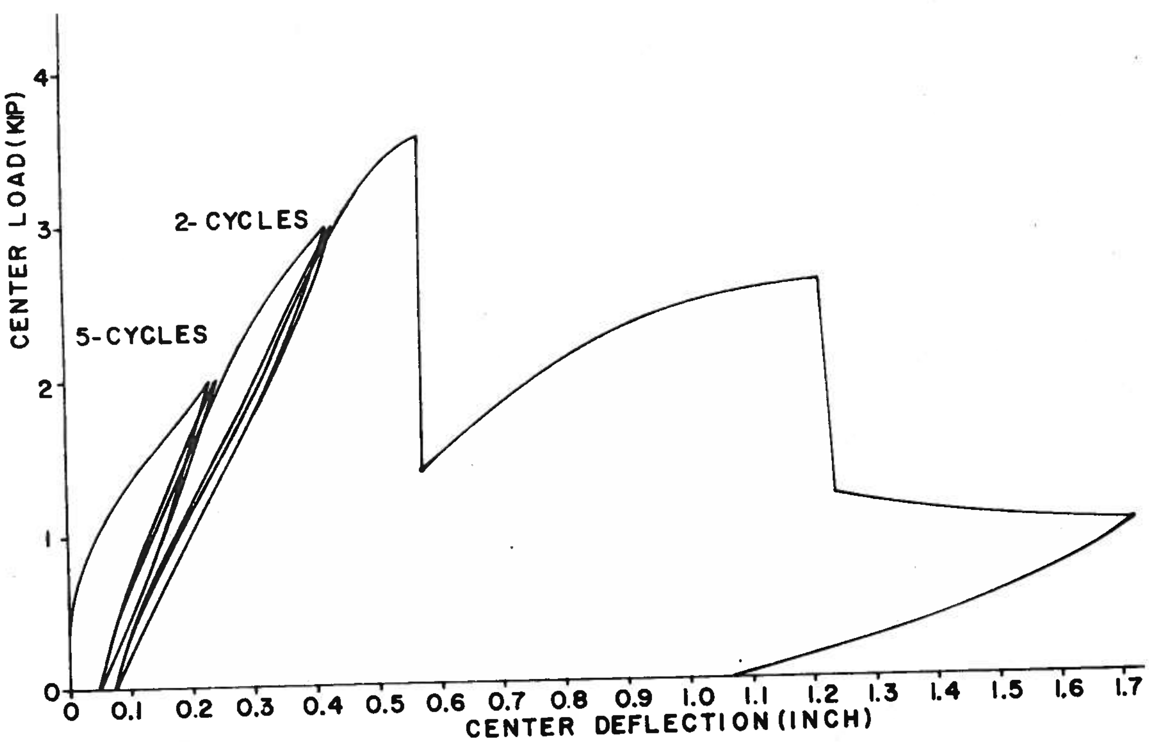


Fig. 23 Load-deflection response of ferrocement, plate-in-compression Specimen 10a made using epoxy and loaded cyclically

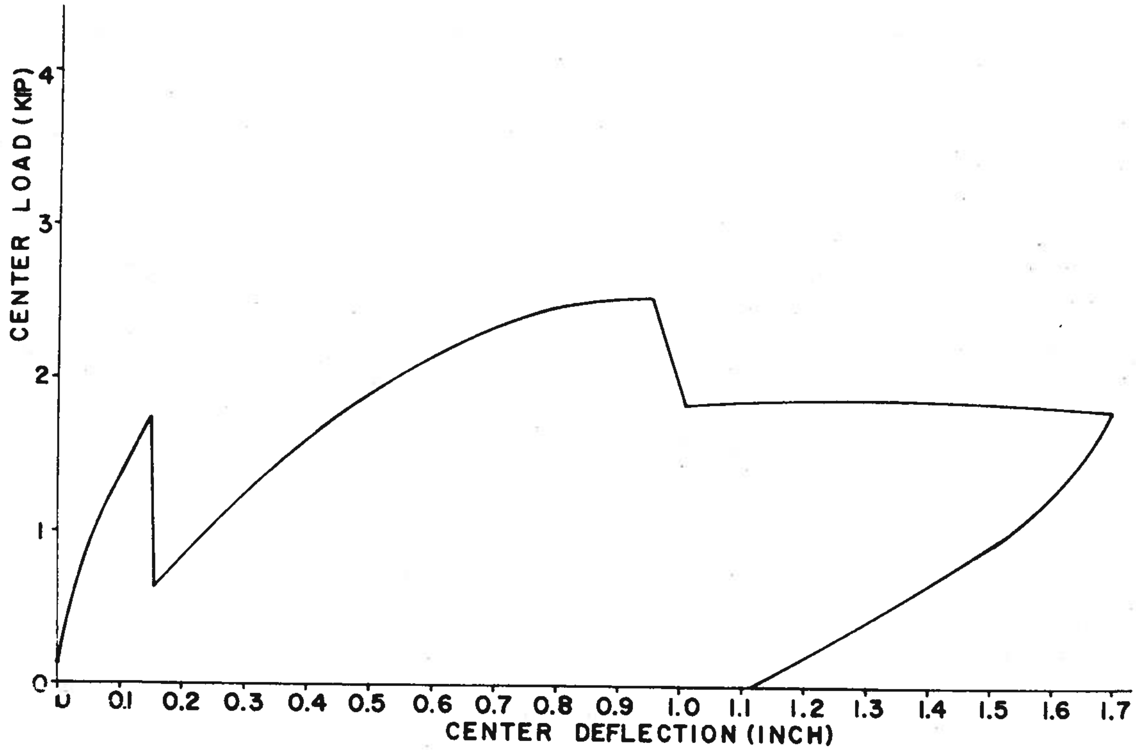


Fig. 24 Average load-deflection response of plate-in-compression Specimens 11 made with 5 layers of mesh and 5 #2 bars and using epoxy

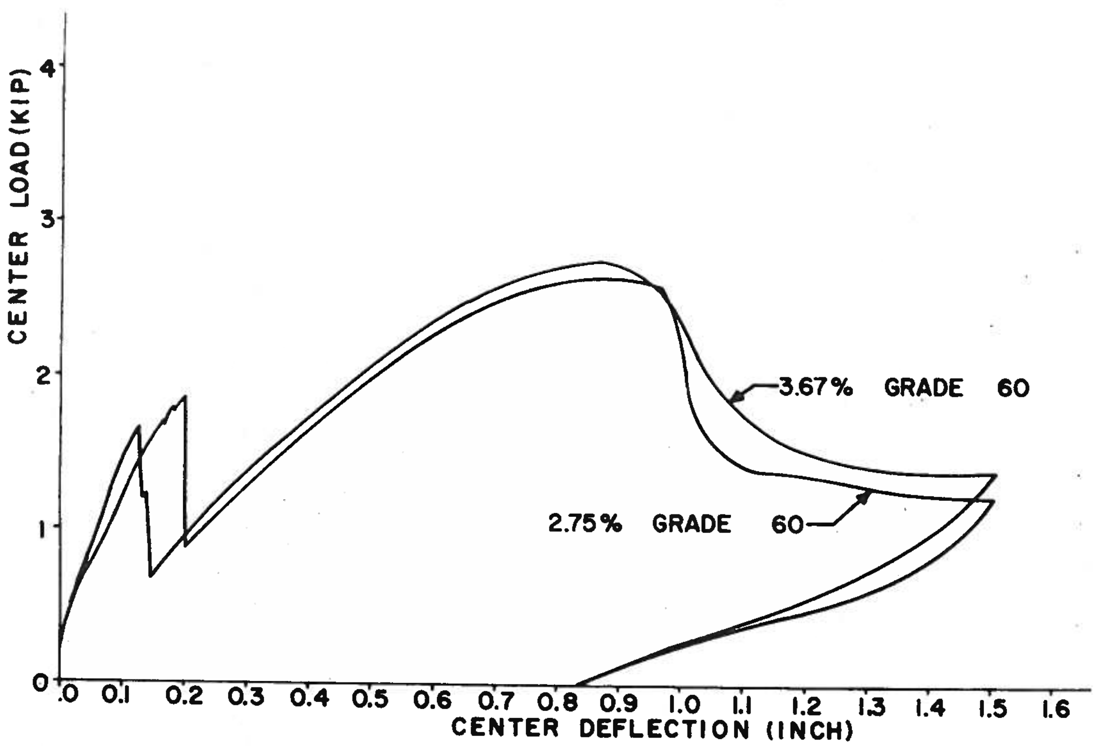


Fig. 25 Average load-deflection response of reinforced concrete plate-in-compression Specimens 12 and 13 made with 3 #3 bars and 4 #3 bars, respectively, and both using epoxy

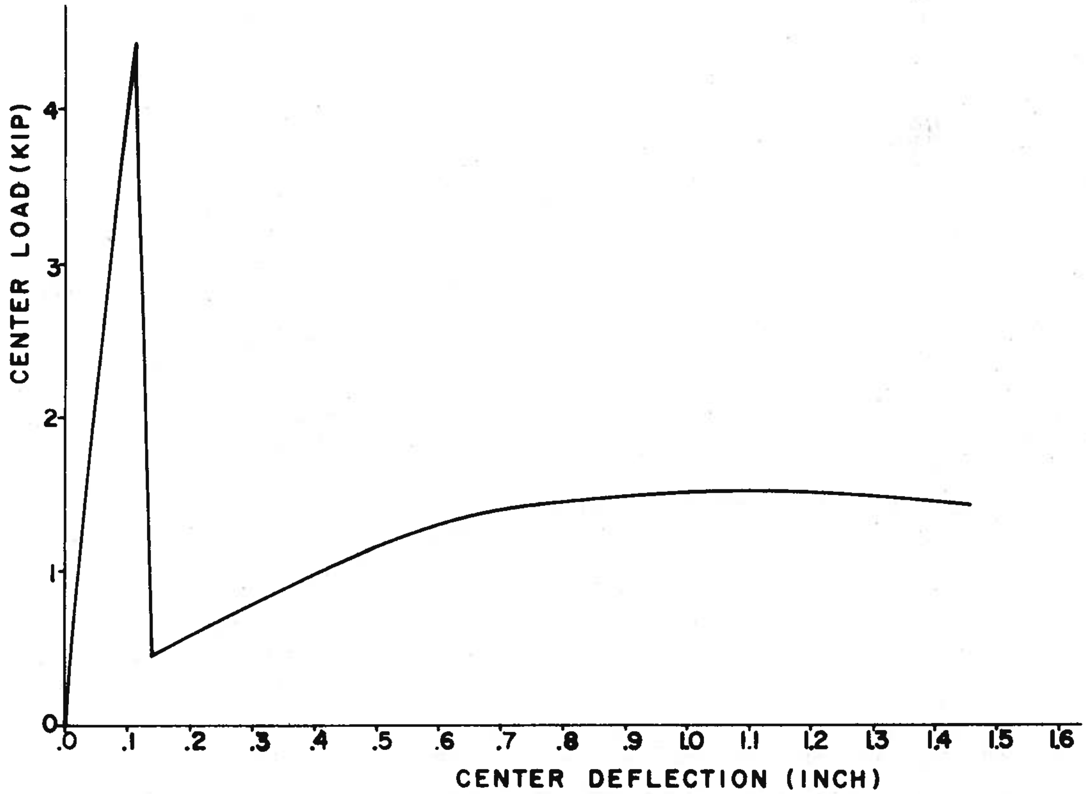


Fig. 26 Average load-deflection response of reinforced concrete, plate-in-tension Specimens 15 made using epoxy

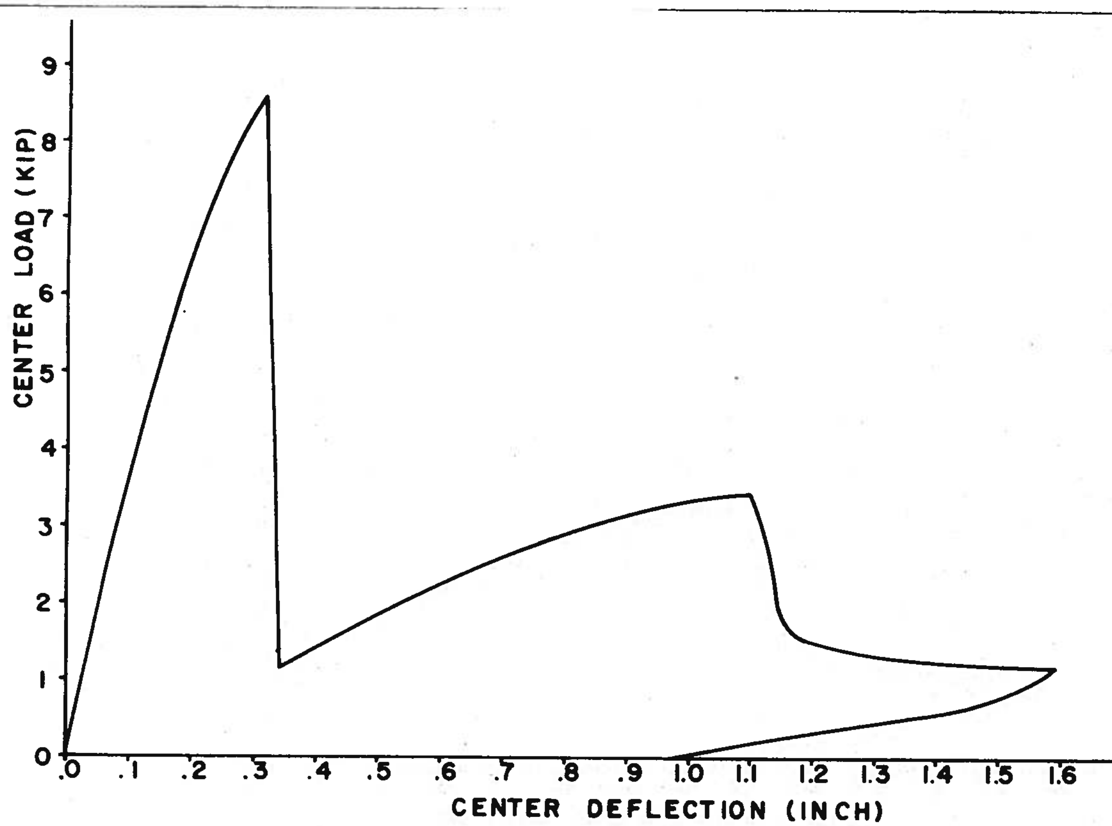


Fig. 27 Average load-deflection response of ferrocement, plate-in tension Specimens 14 made using epoxy and sandblasted plate

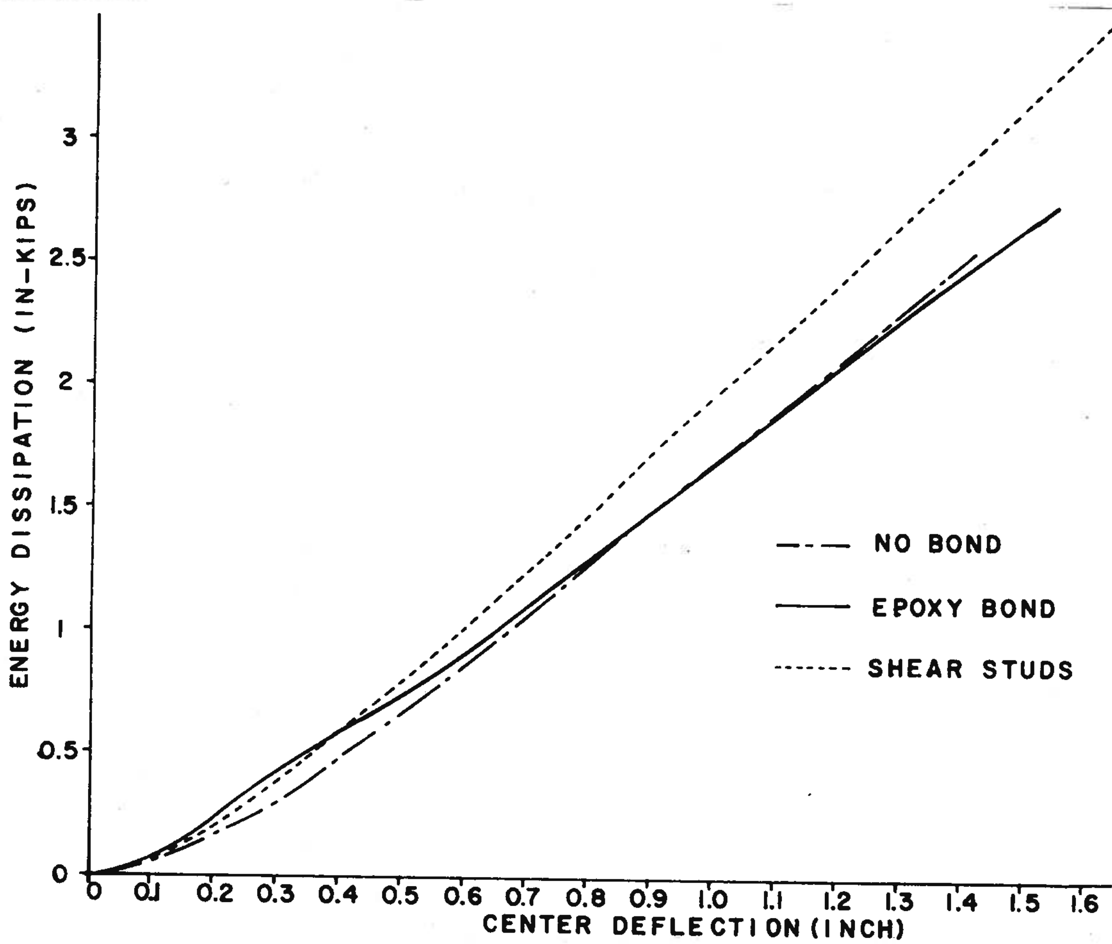


Fig. 28 Energy dissipation of reinforced concrete, plate in-compression composites using various shear connectors

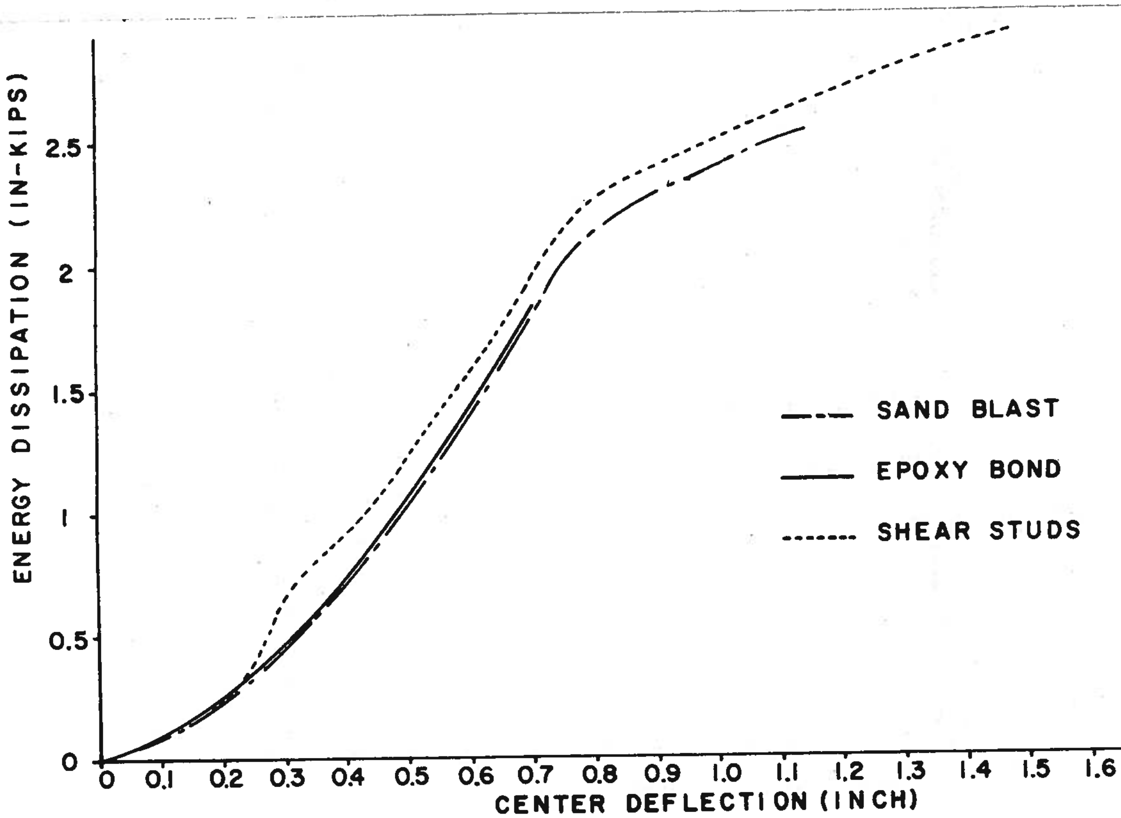


Fig. 29 Energy dissipation of ferrocement, plate-in-compression composites using epoxy, epoxy plus sandblasted plate, and stud shear connections.

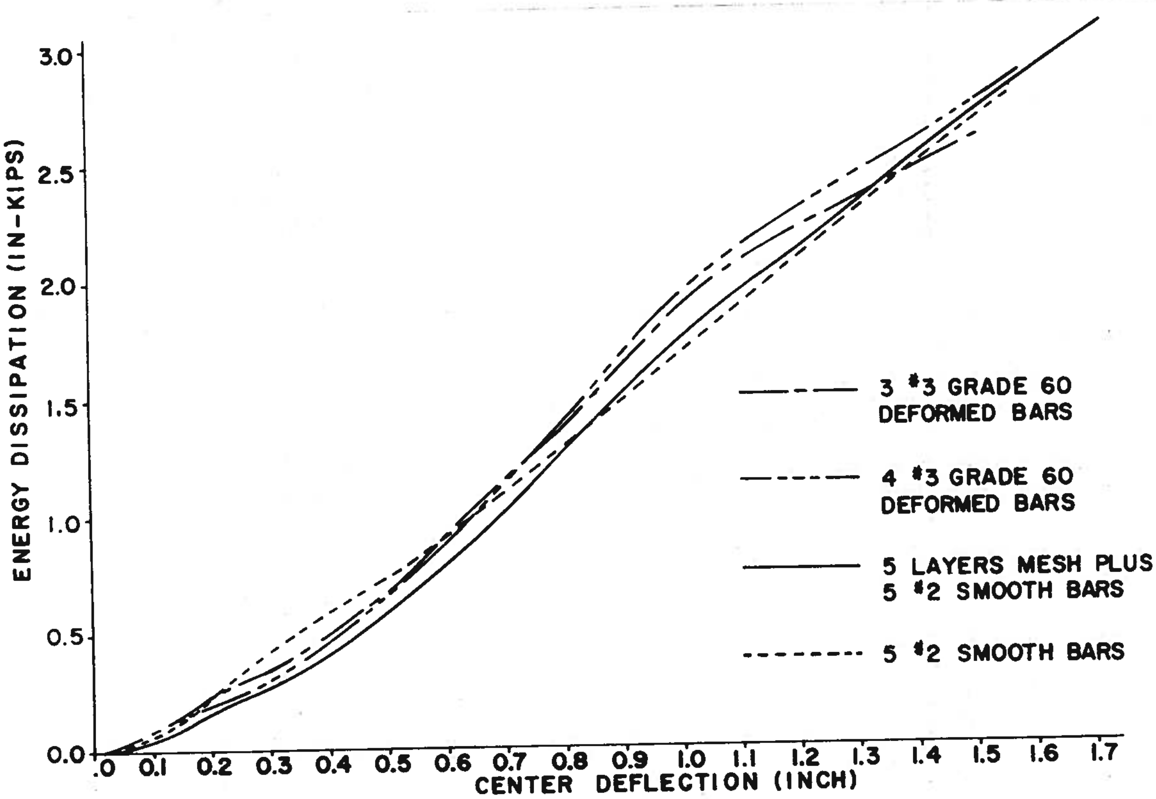


Fig. 30 Energy dissipation of reinforced concrete, plate-in-compression composites using epoxy; made with various amounts of reinforcement.

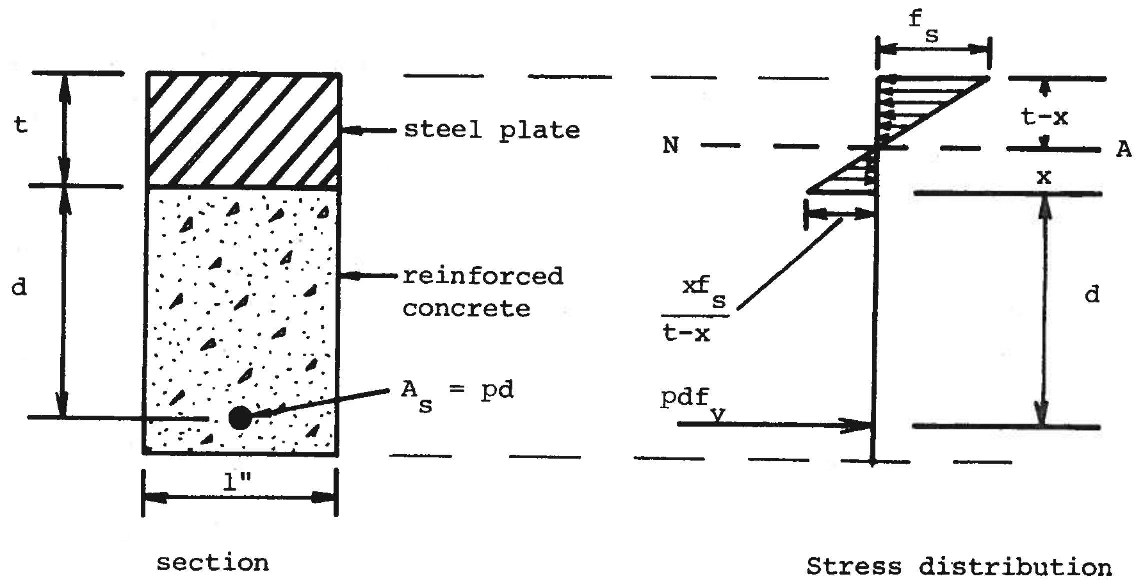


Fig. 31 Stress distribution at yield moment for reinforced concrete with steel plate in compression

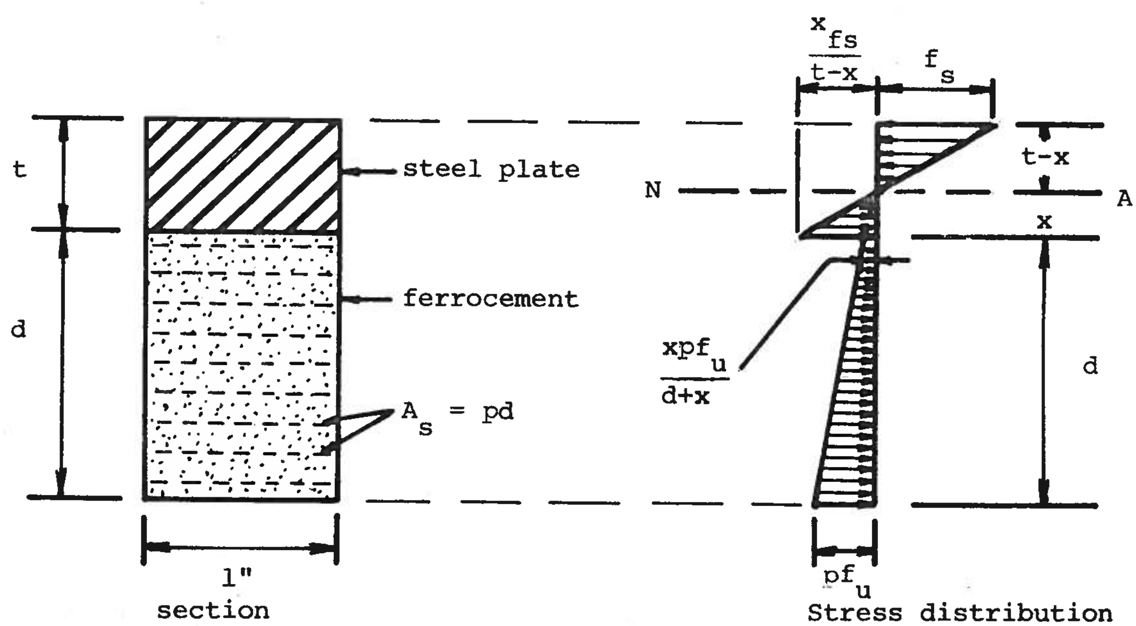


Fig. 32 Stress distribution at ultimate moment for ferrocement with steel plate in compression