AN INVESTIGATION OF FERRO-CEMENT

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I. INTRODUCTION

Ferro-cement generally can be defined as a kind of reinforced concrete made of cement mortar with reinforcement steel in the form of fine wire mesh or expanded metal. The invention of ferro-cement was by the well known Italian engineer and architect Pier Luigi Nervi (10) around the late 1930's.

Nervi explains the development of the idea of ferro-cement as follows: "The fundamental idea behind the new reinforced concrete material, ferro-cemento, is the well known and elementary fact that concrete can stand large strains in the neighborhood of the reinforcement, and that the magnitude of the strains depend on the distribution and subdivision of the reinforcement throughout the mass of concrete. With this principle as a starting point, I asked myself what would be the behavior of thin slabs in which the proportion and subdivision of the reinforcement were increased to a maximum by surrounding layers of fine steel mesh, one on top of the other with cement mortar."

Nervi made use of ferro-cement first in naval construction. Construction of three ships of 400-ton capacity was started in 1943 but stopped by the war. After the war, in 1946 a 165-ton ship was constructed. Following this, several other ships were constructed, all of which have operated successfully.

Ferro-cement has found its real application in civil engineering construction, and again Nervi was the first to use ferro-cement in the construction of a storage shed whose roof and walls were made of 1.1 in. corrugated ferro-cement. Its first important use in civil engineering work was in the construction of the roof of the Milan Fair central gallery in 1947. Since that time a great deal of use of ferro-cement has been made in Italy, especially by Nervi.

Even though ferro-cement has been effectively used in Italy and has been claimed to be a decisive factor both technically and architecturally by Nervi, it has not been used much outside of Italy and not much research has been done on it except for a few recent studies in the 1960's.

There are two main factors which make ferro-cement different from ordinary reinforced concrete. The first is the use of cement mortar made of cement and sand instead of concrete using coarse aggregate. This requires an increase in percentage of cement. In ordinary concrete the cement to aggregate and sand ratio varies between 15 and 18 per cent by weight, while in ferro-cement it is much higher, going up to 60 and 70 per cent. In practice in Italy, 75 pounds of cement have been used per cubic foot of sand (80 per cent). The use of high percentages of cement creates some disadvantages. While an increase in the cement to aggregate ratio increases compressive strength to a certain degree and tensile strength considerably and decreases permeability, it increases shrinkage, creep and surface wear. Since formwork is not always used while making ferro-cement elements, high early strength portland cement has been required.

The second main factor which makes ferro-cement different is the use of reinforcing steel in the form of fine steel mesh or expanded steel sheets. Often the cement mortar is plastered on the steel which has been preformed in place.

In practice in Italy fine steel mesh has been used because it has been the best available steel with fine subdivisions that can be used as reinforcement. Also in Ireland and England expanded steel sheets have been used more recently.

In construction conducted by Nervi, fine steel mesh of ductile steel wires 0.02 to 0.06 inches in diameter set 0.15 inches apart has been used. In naval construction spacing between the layers of mesh has been less according to the conditions. The total thickness of the ferro-cement elements has been a little bit greater than the bulk of the mesh. In later construction to obtain elements of greater strength, one or more layers of steel bars 0.25 inches to 0.4 inches in diameter have been inserted between the layers of mesh. This method has produced thicker and stronger elements having all the properties of the elements formed with the mesh alone.

Because of using reinforcement of fine wires spaced closely, good quality of sand has been required as aggregate and a high percentage of cement has been used. Sand particles having much greater surface area than equal volumes of large aggregate require greater amounts of cement paste to bind them together.

In practice, the cement mortar for ferro-cement is usually placed by hand which requires many more man hours than is required to place normal reinforcing steel. Even though hand placing is counted as one of the disadvantages of ferro-cement, some new techniques such as pumping of concrete or concrete shooting can be used to overcome this.

One great advantage of ferro-cement is that it does not require formwork. Mesh or expanded metal can be put into the required form by using a small amount of scaffolding. Then the mortar is placed by trowelling or spraying. This makes ferro-cement an extremely efficient material to be used in constructions where intricate geometric shapes are used and where wooden forms become too expensive or difficult to make.

Talking about the impressive structure of the Turin Exhibition Hall which is made of ferro-cement, Nervi (10) says, "Obviously a structure of this type built by the usual methods of concrete construction would have required an amount of formwork unobtainable on grounds both of cost and of time while the use of precast reinforced concrete units would have been equally difficult on account of their laborious manufacture and their excessive weight. Ferro-cement, by its independence of formwork, by its intrinsic lightness (the thickness was less than 1.5 in.) was adaptable to the complex problem. I would add that without the constructional qualities of ferro-cement the entire architectural-structural conception would have had to be abandoned or radically changed."

Ferro-cement has found its use also as molds where intricate shapes have been required to be cast in place.

The most important quality of ferro-cement is its greater elasticity and resistance to cracking due to the fine subdivision of reinforcement in the mortar. Decrease in cracking is also aided by the high percentage of cement. In comparison with normal reinforced concrete ferro-cement is much nearer to being a homogeneous material. Since reinforcement is evenly distributed throughout, both steel and mortar will experience the same amount of strain up to a higher strain level than in normal reinforced concrete before cracking. Fine subdivision of reinforcement also increases the shock resistance of ferro-cement considerably so that even after fracture from shock load the mortar will stay in place retaining a certain amount of continuity and some resistance to the passage of water. This property is extremely important for naval use.

Impermeability of ferro-cement is another property which is generally superior over normal reinforced concrete. This property is due to the well known fact that the higher the percentage of cement, the lower the permeability of concrete, and also due to the fact that cracks in concrete are prevented to a large extent by a greater subdivision of reinforcement.

Another characteristic is that it makes possible the use of light and thin elements which reduces the dead weight of structures.

Ferro-cement has been used in different areas successfully. Its use in naval construction as an inexpensive and safe material was cited before. With its high-strength carrying capacity and thin sections, ferro-cement provides a very useful material for plate and shell construction. It is also successfully used in large size pressure pipes. Nervi (10) suggests that it can be used in the manufacture of railway sleepers and in providing a flexible surface for airport runways.

Although ferro-cement has been cited as having certain advantages and useful properties, there has been only limited research to provide sound data about its properties.

This thesis has two objectives. The first is to study the properties of ferro-cement in a comparative way. The second is to explore the possibility of using ferro-cement reinforced with fine mesh reinforcement as flexural members. For this investigation beams reinforced with normal steel reinforcing bars, expanded steel mesh and galvanized wire screen were made using cement mortar (with cement to sand ratio of 20 and 40 per cent) and tested for shrinkage, crack formation, ultimate strength and deflection. The data obtained is used to compare the effects of different reinforcement and different mixes. In addition shallow beams of expanded steel mesh and 40 per cent cement sand mortar were tested to evaluate the general behavior of thin ferro-cement elements. In order to be able to make an economical study, beams of normal reinforced concrete and beams with the bottom parts made of ferro-cement and the top parts made of normal concrete were tested. Tensile and compressive tests on the mortar and on the concrete were also made for comparison.

II. REVIEW OF LITERATURE

The idea of ferro-cement began from the fact that concrete cannot stand large strains in the neighborhood of reinforcement and subdividing reinforcement decreases crack formation.

Investigations have been made on tensile strength of concrete and on flexural cracks in reinforced concrete elements. Since concrete alone is very weak in carrying tensile stresses steel reinforcement must be used. When the steel in the tension zones is stressed beyond a certain limit, concrete around it cracks. The excessive formation of cracks increases the danger of corrosion of the reinforcing steel. Especially in structures where water tightness is required, cracks are undesirable.

The historical background on crack studies can be traced as was done by Michael Chi and Aphur F. Kirstein (1). In 1936 two papers on crack formation were published by Barnemann (2) and by Colonnetti (3). Barnemann concluded that the width of cracks in concrete increases as the maximum tensile force increases up to rupture and the width of cracks decreases as the ratio of steel area to concrete area increases. Colonnetti (3), making a theoretical analysis, concluded that the width of cracks decreases as the diameter of the reinforcing bars decreases. A similar conclusion was reached by Watstein and Parsons (4) who stated that the most important factor affecting crack width and crack spacing is the ratio of diameter of bars to the area of concrete. They conclude that crack width is almost independent of the strength of concrete. Making some studies on the effect of bond efficiency of reinforcing bars on the width and spacing of cracks, Watstein and Seese (5) conclude that as the bond efficiency increases the width of the cracks decrease and that width of cracks and spacing of cracks are linearly related.

In 1948 Byuggren (6) reached the conclusion that the spacing of cracks in a beam is influenced by the smallest surrounding area of concrete. In the same year Wastlund and Jonson (7) made some tests on T-beams with varying dimensions and steel ratios and arrived at the following conclusions:

1. For a given stress level in the reinforcement, crack width increases almost linearly with bar diameter for beams of various dimensions and with various amounts of reinforcement.
2. For a given cross section of concrete and a given diameter of reinforcing bars, the width of cracks at a specified steel stress decreases only slightly even when the area of the steel is increased threefold.

3. All other variables being equal, wider beams have considerably larger crack widths while an increase in beam depth has practically no effect on the width of the cracks.
4. Crack width decreases with an increase in the surface roughness of the reinforcing bars.
5. Variation of the concrete strength did not exert any noticeable influence on the width of cracks.

A. Clark (8), improving an equation given by Watstein and Parsons (4) obtained the following equation:

$$w = 2.27 \times 10^{-8} \left(\frac{h-d}{d}\right)\left(\frac{D}{p}\right) \quad fs - 56.6\left(\frac{1}{p} + 8\right)$$

where

w = average width of cracks

h = overall depth of beam

d = distance from steel to the compressive face

D = diameter of the bars

p = area of steel/area of concrete

f_s = stress in the steel

The investigations that he made are in qualitative agreement with the findings of other investigators who report that the width of cracks can be reduced by using a large number of small bars and by increasing the ratio of reinforcement.

Michael Chi and Arthur F. Kirstein (1) arrived at the same general conclusion with some additions, saying that "the minimum average spacing of cracks was found to be proportional to the product of the diameter of the reinforcing bars and the parameter ø (the ratio of the assumed effective area to the fully developed area of concrete) which was dependent on the general arrangement and diameter of reinforcement." They also stated that variation in concrete strength in the range from 2000 to 6000 psi had practically no effect on the formation of cracks.

M. F. Kaplan (9) has investigated the effect of composition of concrete mix on cracking, and he has arrived at the following conclusions: "Tensile strains and stresses at cracking under load depend on the volume of coarse aggregate in the mix. The greater the volume of coarse aggregate the lower the strain and stress at cracking. This may be due to internal strain and stress concentrations caused by the coarse aggregate inclusion."

In the experiments (10) carried out in Italy by Professor Oberti in 1947 to obtain some numerical data on the degree of straining of ferro-cement without visible cracking similar results were obtained as in the early investigations by Bornemann (2). The tests by Oberti show the vital importance of the ratio of steel to concrete quantities in strain limitation of concrete without cracking. With steel mesh weight of 220 lbs. to 440 lbs. per 35 $ft.^3$ of concrete, that is 4.2 to 8.4 per cent steel by weight, the strain limitation remained approximately the same as that of an unreinforced mortar. With an increase in the weight of mesh up to 880 lb. to 1100 lb. per 35 $ft.^3$ of concrete, that is 16.8 per cent to 21.0 per cent steel by weight, the strain limitation has jumped up to four or five times the first case.

The first known paper dealing directly with ferro-cement reports on some experiments for which complete data is published by Lyal D. G. Collen (11). A set of tests of simply supported beams was carried out to determine the bending strength, shear strength and modulus of elasticity of ferro-cement with differing cement-sand and water-cement ratios. The beams tested contained several layers of machine woven mesh of non-galvanized 18 standard wire gage steel closely spaced with a layer of 1/4 rods of steel in the middle. Expanded metal was also used in place of mesh and differences were discussed.

The following results were reported from the tests:

1. The bending strength is proportional to the steel content.
2. Shear strength is almost linearly proportional to the steel content.

3. Considering ferro-cement as a homogeneous material, Young's modulus is related to the steel content almost linearly.
4. Increase in the water-cement ratio decreases the ultimate bending strength.
5. Increase in cement-sand ratio up to 0.65 by weight causes increase in ultimate bending strength.
6. A further increase in cement-sand-ratio causes a decrease in strength.

Collen concluded that a water ratio of 0.35 will be reasonable to use, giving adequate strength and sufficient workability. The high cost of wire mesh initiated a search for other kinds of mesh that could be used. Because of its lower unit cost and a smaller labor requirement expanded metal was investigated. A graph relating the ultimate bending strength to the cost of steel showed that expanded metal was more economical than wire mesh.

Some tests have also been run on trough sections made of ferro-cement. The results show that the same bending strength as in the normal reinforced concrete elements can be obtained with ferro-cement with much less dead weight. Collen suggests prestressed ferro-cement elements to be used in longer spans.

The fact that an increase in the cement-sand ratio after a certain limit decreases the strength of concrete is also referred to by Neville (12) who explains that, as the cement-sand ratio goes higher and higher the percentage of water in the volume increases, increasing air voids which cause a decrease in strength.

A second paper on ferro-cement by J. G. Byrne and W. Wright (13) presents an investigation carried out to verify the use of expanded metal for a ferro-cement roof canopy. In the investigation a cement-sand ratio of 0.7 by weight and a water-cement ratio of 0.4 has been used. Shrinkage tests on mortars of different ratios have also been reported.

They found no disadvantage in the use of expanded metal but found it advantageous for providing good mechanical bond and ease of placing since fewer layers were needed. One important point they make is that the mortar in some cases split badly due to the scissor action of the diamond mesh and thus put a limit on the size and weight of mesh that could be used. Mesh reduces the deflection due to shrinkage since it is distributed in a uniform manner. This provides a good solution to the deflection due to shrinkage in structures made of thin light elements.

The high cement-sand ratio has at least one advantage and some disadvantages. An advantage is that it increases the impermeability of the mortar due to the fact that hardened cement paste has extremely fine texture so that the pores are very small and numerous while in sand, although pores are fewer, they are much larger and there is higher permeability. Because of this, as the sand volume decreases, impermeability increases.

Disadvantages include the high shrinkage and creep rates. Some information is found in Neville's (14) book about variation of shrinkage with the variation of mix proportions in concrete. The main cause of shrinkage is pointed out as the loss of adsorbed water in concrete and the most important factor affecting shrinkage is pointed out as aggregate content. The ratio of shrinkage of concrete, S_c, to shrinkage of neat paste (S_p) depends on the aggregate content, a, and is expressed as

$$S_c = S_p(1-a)^n$$

where n is a constant determined experimentally and varies between 1.2 and 1.7 (15) due to relief of stresses in the cement paste.

Neville does not believe that water content is a primary factor in shrinkage because the change in the volume of drying concrete is not equal to the volume of water removed. The loss of free water which takes place first causes little or no shrinkage because this results in emptying the capillaries which stay as voids afterwards. Since with emptying of capillaries the voids do not close up and disappear, no change occurs in the texture of concrete so the volume stays the same. Once the capillary water has been lost the removal of adsorbed water takes place. The adsorbed water surrounds the cement particles and sets the center of particles farther apart. When the loss of adsorbed water starts, the cement particles tend to move toward each other, decreasing the total volume of concrete. This happens to be the cause of shrinkage due to water loss.

III. PROGRAM OF EXPERIMENT

A. Materials Used.

Mortar used in the test was made of cement and sand. The sand grading was held constant in all tests, according to the following analysis:

Sieve Size	200	100	50	30	16	8
Percent Passing	1.5	5	8	40	80	100

Cement used was high early strength portland cement with air entrainment (Type III-A).

Three different kinds of reinforcement were used.

1. Number 2 plain steel bars.

   Diameter = 1/4 in. per bar

   Area = 0.05 $in.^2$ per bar

   Weight = 0.167 lb. per foot

   Yield stress = 55 ksi

   $E = 30.4 \times 10^3$ ksi

   Values of yield stress and E are calculated from the data obtained on direct tension test.

2. Expanded steel lath.

   Thickness of the sheet = 0.017 in.

   Width one subdivision = 0.05 in.

   Effective area per inch of actual width = 0.00484 $in.^2$

Weight per square foot = 0.281 lb.

Ultimate breaking load = 37.4 k/in.^{2}

3. Galvanized steel wire screen.

Diameter per thread = 0.011 in.

Area per thread = 0.000098 in.^{2}

Number of threads per inch of longitudinal direction = 19

Area per inch length of screen = 0.00186 in.^{2}

Weight per square foot = 0.133 lb.

Ultimate breaking point = 102 k/in.^{2}

B. Preparation of Beams.

Beams were cast into steel forms two at one time. The form was 100 in. long, 7 in. deep and 4 in. wide. A wooden partition was placed in the middle of the form to separate two beams. Brass plugs, which were to be used to measure strains in the beams were connected to the form tightly so that they would be embedded in the concrete after the beams were cast. Figure 1 shows the location of plugs. The form was oiled before each casting operation to make the removal of the form easier.

After the casting operation finished the beams, they were put into the humidity chamber to accomplish a good curing process. The form was removed after 24 hours and the beams were kept in the humidity chamber for a total of eight days.

In all mortar mixes, the water-cement ratio was held constant at 0.35. Mortar was mixed in a 2 ft^3 electrically operated rotary mixer. Since the placing of concrete took up to four hours, the mixer was continually revolved to prevent settlement and separation of the concrete. When reinforcing bars were used they were hooked at the ends to provide anchorage.

Expanded steel lath and galvanized wire screen were cut into strips 3.5 in. wide and placed alternately with layers of mortar.

In the case of beams with steel bars, a vibrator was used to compact the mortar.

In the case of expanded steel lath and galvanized wire screen, a layer of mortar was poured and compacted by placing a piece of board on it and hammering. Then a layer of reinforcement was put and another layer of mortar was poured. Pieces of wood were used to keep spacing between layers of reinforcement consistent. The position of the different types of reinforcement are shown in Figures 2, 3, 4 and 5.

C. Testing Procedure.

The following tests were performed for each mix:

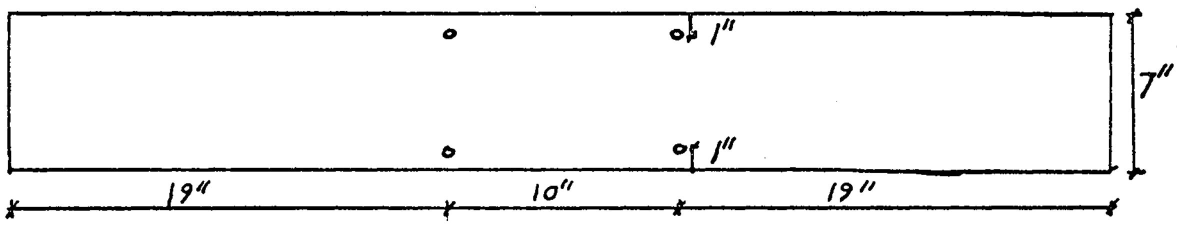

Figure 1. Location of Brass Plugs on the Casting Form.

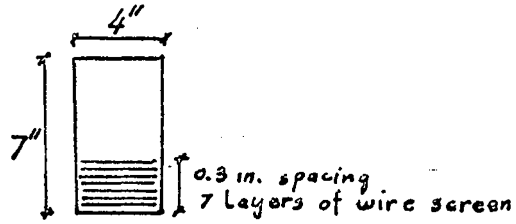

Figure 2. Cross-section of a Beam with Galvanized Wire Screen.

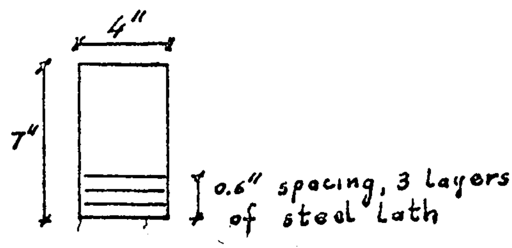

Figure 3. Cross-section of a Beam with Steel Lath.

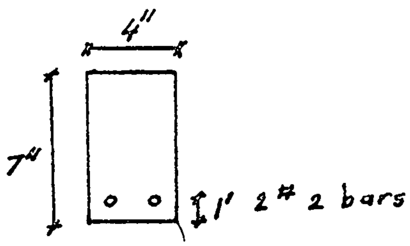

Figure 4. Cross-section of a Beam with #2 Plain Steel Bars.

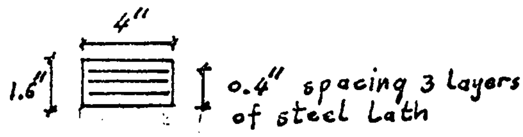

Figure 5. Cross-section of a Thin Beam.

1. Shrinkage Test.

The purpose of the shrinkage test was to record the shrinkage that might have occurred in the beams during the first 20 days after they had been removed from the humidity chamber. The beams were kept on a flat floor without end restraints or applied loads. Shrinkage measurements were taken at the top and at the bottom of each beam making use of brass plugs that were placed in the side of the beams. Steel pins with machined center holes to receive the Whittemore mechanical strain gage were screwed into the plugs. Readings on the strain gage permitted the calculation of strain to the nearest 0.00001 inches per inch.

Readings were taken on 1st, 3rd, 6th, 10th and 19th days after the beams were taken out of humidity chamber. During this time beams were checked to see if there were any visible crack formations which might have been due to shrinkage.

2. Beam Tests.

The purpose of these tests was to record the top and bottom fiber strains, center deflection and occurrance of visible cracks at different loads and to find the ultimate strength of each beam when simply supported and subjected to third point loading. The beams were tested with Tinus Olsen mechanical straining beam testing machine with a maximum capacity of 100,000 pounds.

The loading condition for all beams during testing was as shown in Figure 6. Beams were loaded by increasing the magnitude of P at intervals of about 200 pounds in the early stages of the tests and about 100 pounds when the deflection rate increased. At each increment of load strains were measured with Whittemore mechanical strain gage at the same locations as for the shrinkage measurements. Center deflections to the nearest 0.001 inches were measured using a dial gage. Also at each increment of load, the beams were carefully inspected to note the formation of visible cracks. Loading was continued until the beams yielded excessively at ultimate load.

3. Compression Test.

The purpose of this test was to determine the compressive strength of each mortar mix. Two 6 x 12 in. cylinders of each mix were tested on a Tinius-Olsen 400,000 pound capacity testing machine.

4. Tension Test.

The purpose of this test was to determine the tensile strength of each mortar mix under direct tension. Three brickets from each were tested to failure on a Soil test, type C.W. machine.

5. Tests on Reinforcing Steel.

Number 2 plain steel bars were tested in tension to obtain the stress-strain relationship and the

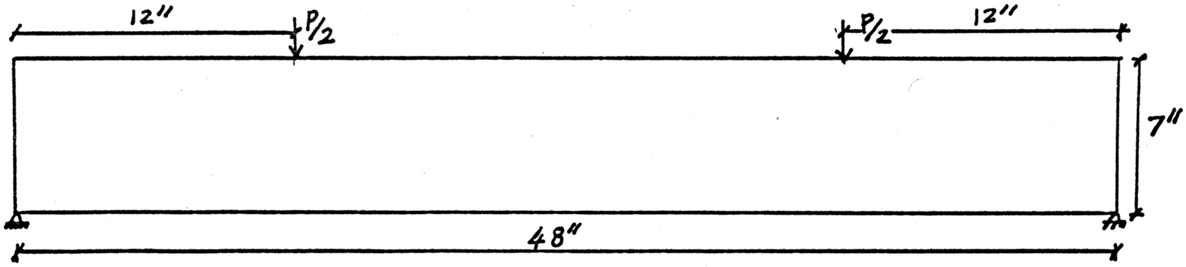

Figure 6. Loading Condition of the Beams.

Figure 7. Applying Load to a Beam.

Figure 8. Measuring Strains with the Whittemore Strain Gage.

yield stress. Direct tension was applied to the bars with a Tinius-Olsen 120,000 pound testing machine.

In the case of galvanized steel wire screen and expanded steel lath, it was very difficult to find stress-strain relationship. When these were tested in tension with Tinius-Olsen testing machine, they experienced large initial strains at low loads. This was due to the loose texture of both the wire screen and the expanded steel. It was not possible to measure the ultimate tensile strength of these by this method so pieces of expanded steel and wire screen were embedded in brickets and tested in tension. In the middle portion cardboard pieces were inserted across the concrete section to allow all the load to be carried by the expanded steel or wire screen.

D. Beam Description.

Information about different kinds of beams are given in Table 1.

Beams 1-12 and 15-18 were 4 in. wide, 7 in. deep, and 48 in. long. Beams 13-14 were 4 in. wide, 1.6 in. deep and 48 in. long.

Beams 17-18 were made of two different concrete mixes. The part from the bottom up 2.4 inches was made of sand-cement mortar as properties given in Table 1. The upper part was made of normal concrete with properties as given in the table.

Table 1. Beam Properties

Beam No.	S/A	C/A	W/C	Kind of Reinforcement	A_S/A_C	Density of Concrete	f'c (psi)	f't (psi)	fy (psi)
1 2	1.00	0.40	0.35	#2 bars	0.00357	143	7500	860	55
3 4	1.00	0.40	0.35	Steel lath	0.00181	141	7348	895	37.4
5 6	1.00	0.40	0.35	Wire screen	0.00163	140	7150	790	102
7 8	1.00	0.20	0.35	#2 bars	0.00357	115	2700	225	55
9 10	1.00	0.20	0.35	Steel lath	0.00181	114	2460	183	37.4
11 12	1.00	0.20	0.35	Wire screen	0.00163	114	2310	230	102
13 14	1.00	0.40	0.35	Steel lath	0.0080	146	7120	775	37.4
15 16	0.35	0.18	0.50	#2 bars	0.00357	149	3140	-	55
17* 18*	T:0.35 B:1.00	T:0.18 B:0.40	T:0.50 B:0.35	Steel lath	0.00181	T:144 B:150	T:7050 B:5360	- -	37.4

* Values given are for top (T) and bottom (B) of beams.

Notation for Table 1.

A_s/A_c = ratio of area of steel to gross area of concrete

S/A = ratio of sand to total aggregate ratio (by weight)

C/A = ratio of cement to aggregate (by weight)

= cement/sand ratio where S/A = 1

W/C = ratio of water to cement (by weight)

f'c = concrete cylinder strength

f't = mortar tension strength

fy = yield strength for bars; or breaking strength for lath and screen

Each pair of identical beams was cast from a different mix. From each mix samples were prepared for direct compression and tension tests. Compression specimens were 6 x 12 in. cylinders. Tension specimens were cast in small bricket molds.

After the specimens were cast they were placed in a humidity chamber for eight days and then kept in normal room atmosphere until testing. The forms were removed 24 hours after casting.

E. Shrinkage Results.

The results shown in Table 2 are the averages of shrinkage measurements on two beams of each type.

A comparison of the shrinkage rate in mixes with sand/cement ratio equal to 0.20, 0.40 and in normal concrete is shown in Figure 9. The data plotted is that obtained from upper set of gage plugs on beams No. 1, No. 7 and No. 13.

F. Test Results.

The loads at which cracks formed and the ultimate load capacity of each beam is given in Table 3.

All of the beams were under-reinforced so that ultimate loads were reached when the steel bars yielded or when expanded steel lath or steel wire broke.

In the case of beams 3 and 17 initial vertical cracks were present before loading. During loading these cracks grew wider and caused failure.

Table 2. Average Total Shrinkage in Beams (micro inches/inch)

Beam Number	Reinforcement	Cement / Sand	Gage Location	Days* 1	3	6	10	19
1-2	Bars	0.40	Upper	0	90	330	500	600
			Lower	0	80	310	490	590
3-4	Lath	0.40	Upper	0	100	350	540	650
			Lower	0	70	290	430	530
5-6	Screen	0.40	Upper	0	90	330	510	610
			Lower	0	80	290	450	550
7-8	Bars	0.20	Upper	0	70	200	320	370
			Lower	0	60	190	310	360
9-10	Lath	0.20	Upper	0	60	200	320	410
			Lower	0	50	160	250	300
11-12	Screen	0.20	Upper	0	50	200	310	380
			Lower	0	50	150	230	290
13-14	Bars	0.18	Upper	0	30	80	150	210
			Lower	0	20	70	150	200
15-16	Lath	0.40	-	0	90	290	470	570
17-18	Lath	0.18	Upper	0	40	100	170	230
			Lower	0	80	280	420	510

*Days after moist curing for 8 days.

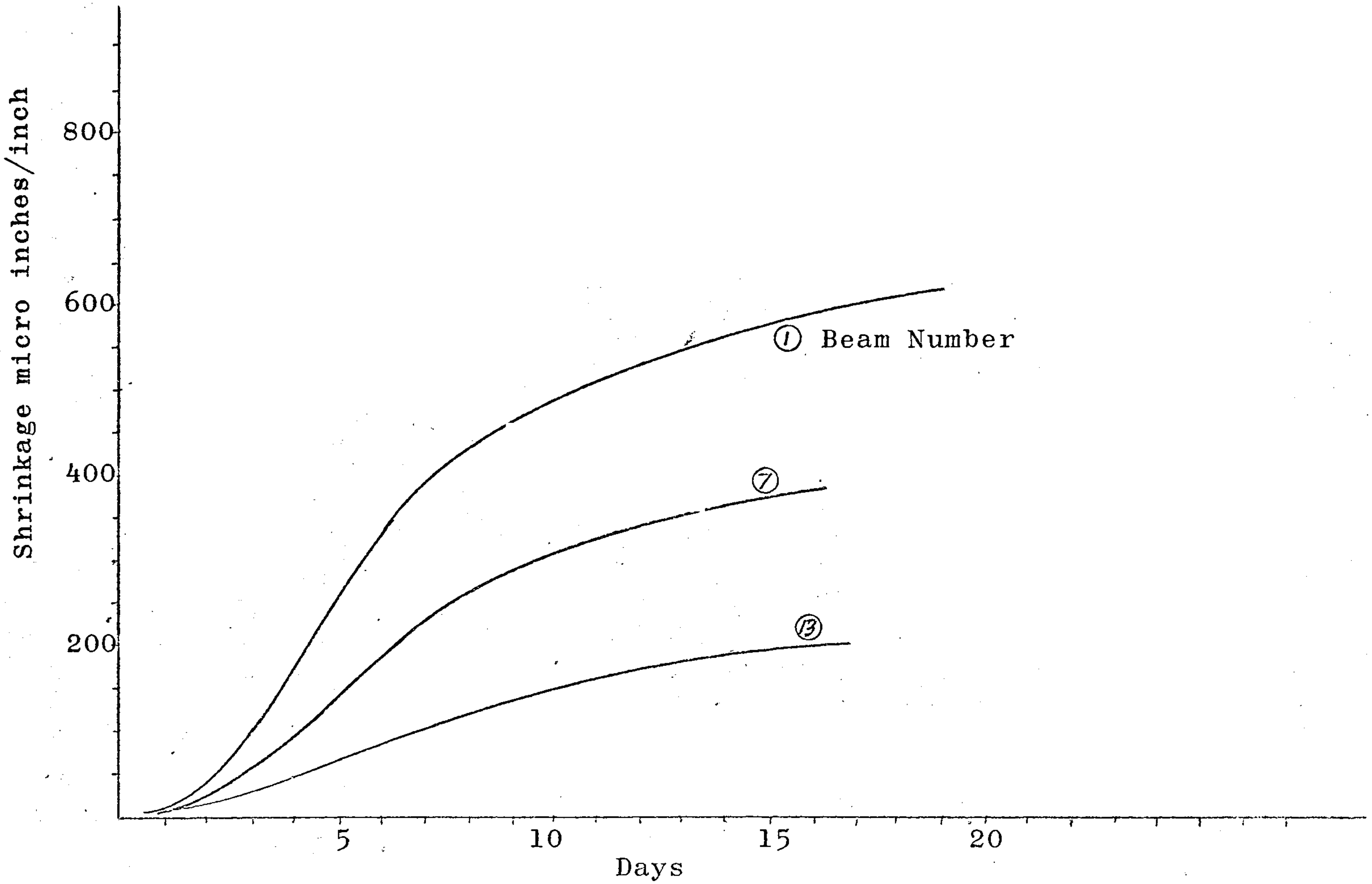

Figure 9. Shrinkage Curves.

Table 3. Crack Forming Loads and Ultimate Load Capacities of Beams

Beam	First Crack Load (lb.)	Second Crack Load (lb.)	Third Crack Load (lb.)	Fourth Crack Load (lb.)	Ultimate Load (lb.)
1	3850	4200	-	-	4870
2	3550	-	-	-	4665
3	Initial Crack	-	-	-	1474
4	1848	1896	-	-	1926
5	3150	-	-	-	3185
6	3214	-	-	-	3295
7	450	857	1660	2033	2053
8	674	1517	-	-	2094
9	750	750	-	-	794
10	715	750	715	-	775
11	1242	1260	-	-	1295
12	Initial Crack	-	-	-	788
13	134	169	173	-	200
14	164	-	-	-	185
15	2409	3070	3737	-	4310
16	2390	2577	-	-	4150
17	Initial Crack	-	-	-	1290
18	1770	-	-	-	1823

In the case of beam 12 an initial horizontal crack was present which caused shear failure of the beam.

In the case of beams 13 and 14 the load capacity flucuated up and down.

The crack patterns of the beams are shown in Figure 10.

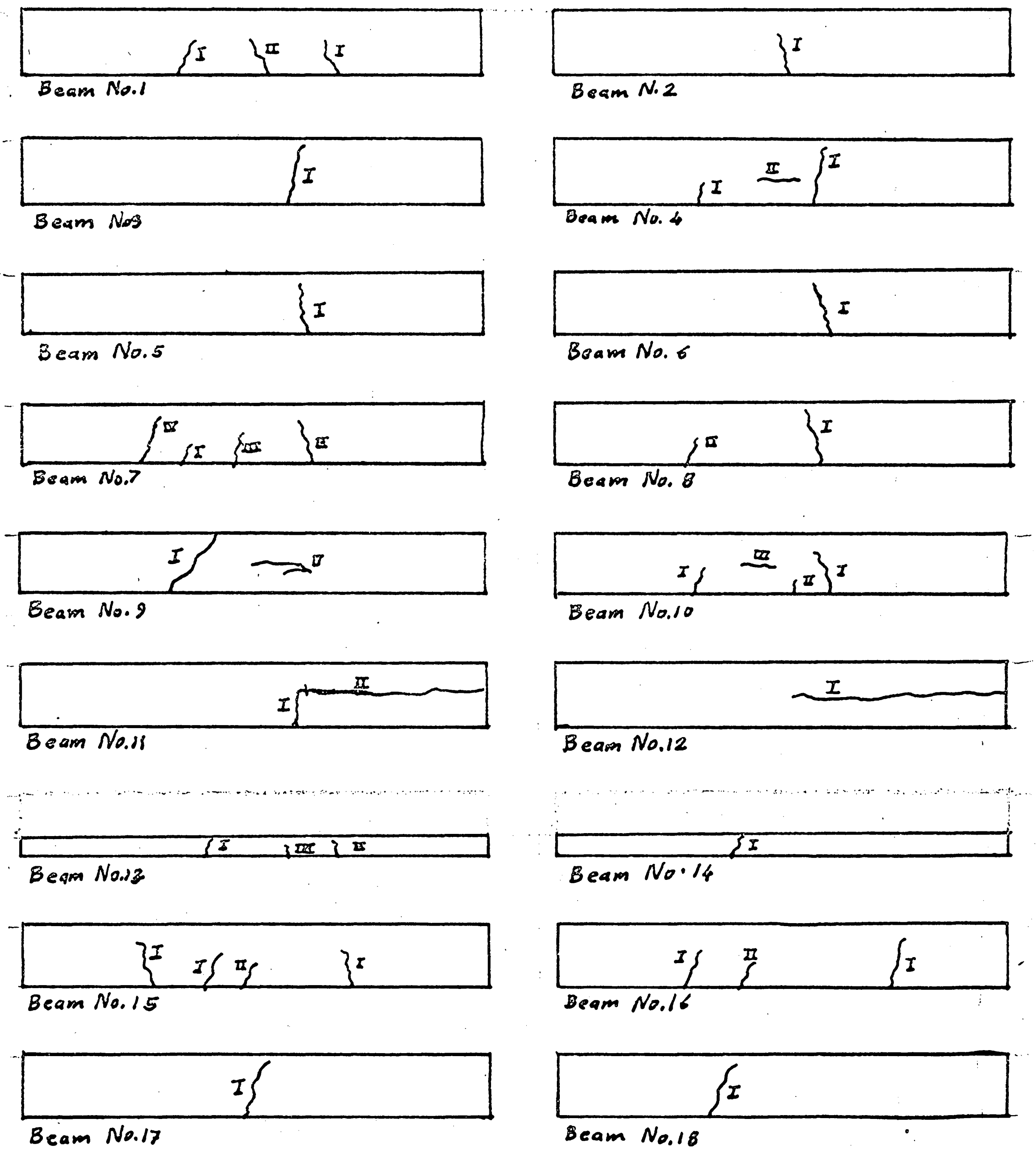

Figure 10. Crack Patterns.

IV. ANALYSIS OF RESULTS

A. Shrinkage Analysis.

The shrinkage data obtained confirms the information given by Neville (14) that shrinkage increases as the cement to aggregate ratio increases. In mixes where cement to sand ratio was 0.40, the shrinkage measured after 19 days from the time beams were taken out of humidity chamber was about 600-650 micro-inch inches, while in mixes where cement to sand ratio was 0.20 the shrinkage measured at the end of 19 days was about 350-400 micro-inch inches.

In the mixes where cement to aggregate ratio was 0.18 and where coarse aggregate along with sand was used, the shrinkage measured after 19 days was about 200-250 micro-inch inches. In the beams with cement to sand ratio equal to 0.20 and where only sand was used as aggregate the shrinkage was about 350-400 micron-inch inches as mentioned above. The increase in shrinkage in beams with cement to sand ratio equal to 20 may be considered partly due to the slight increase in the cement to sand ratio and may very well be partly due to the use of sand only without any coarse aggregate. Even though Neville (14) states that gradation of aggregate does not affect shrinkage, there is not complete agreement on this in the literature. J. C. Saemann and C. Warren (16) in their paper on shrinkage give gradation of aggregate as a major factor affecting shrinkage.

The shrinkage in the beams with cement to sand ratio equal to 0.40 was about 600-650 micro-inch inches at the end of 19 days. This value is quite high and may cause shrinkage cracks in members whose ends are restrained against movement. In these tests initial cracking during moist curing were noted in only two beams. Shrinkage cracks were not expected since these beams were not restrained at the ends and shrinkage normally does not occur at this time. It is possible that the cracks in the two beams were caused by premature loading from handling or lack of uniform support because of uneveness in the floor.

An interesting point is the fact that the shrinkage of the lower fibers is more noticeable than that of the upper fibers on the beams with expanded steel lath and steel wire screen by about 100 micro-inch inches. This is attributed to the fact that the reinforcement is distributed uniformly across the cross section of the beam and restraints the shrinkage uniformly. In the cases where a higher steel ratio is used shrinkage may be cut down considerably and may not appear as a problem.

B. Crack and Concrete Strength Analysis.

The values of compressive cylinder strengths and tensile strengths of mortar used in the tests are given in Table 1.

The values of both compressive strength and tensile strength for mortar with cement to sand ratio equal to 0.20 are quite low. This is attributed to the fact that since sand has a much larger surface area to volume than coarse aggregate, it requires a larger amount of cement paste to bind the sand particles together. Concrete containing coarse aggregate and having a cement to sand ratio equal to 0.18 has much greater strength in both tension and compression than the mortar with C/S equal to 0.20 because of this fact.

The mortar with cement to sand ratios equal to 0.40 obtained very high strength both in compression and tension, which showed that in this case cement paste volume is sufficient to bind the sand particles together. The high strength obtained is also attributed to the low water to cement ratio which is 0.35.

The difference in cement to sand ratios greatly influenced in general the load bearing capacities of the beams as shown in Figures 13-18. The beam with C/S ratio equal to 0.20 carried 57 per cent less load in case of beams with plain bars, 46 per cent less in case of beams with expanded steel lath and 59 per cent less in case of beams with steel wire screen than the beams with C/S ratio equal to 0.40. Also there was a difference of 11.5 per cent between the ultimate bearing capacities of beams 1-2 and beams 15-16.

These results can be explained by considering the stress-strain relationship of concrete. The following equation was obtained by Desayi and Krishnan (17):

$$f = \frac{E_e}{1 + \left(\frac{e}{e_o}\right)^2}$$

where

f = stress at any strain e

e_o = strain at the maximum stress of f_o

E = $f_o/e_o/_2$ = a constant

$k_1 f_o$ = stress at failure

e_c = maximum strain at failure

f_o = $k_2 f'c$

$f'c$ = cylinder strength of concrete

To get the total compressive force in concrete, the stress equation can be integrated between zero and the strain of the upper fiber, e_c and multiplied by the beam width. The result is:

$$F_c = \tfrac{1}{2} E\, e_o^{\,2} \log e \left(1 + \frac{e_c^{\,2}}{e_o^{\,2}}\right) \frac{1}{e_c}\, k_{nd}\, b$$

where

k_{nd} = distance from the compression side to the neutral axis

b = width of the beam

As is seen from the above equation E, which is a function of f'c, is directly related to the total compressive force. Since in two beams with different concrete strength and with the same kind of reinforcement at a certain cross section with the same bottom fiber strains the tensile forces are the same, the compressive forces must also be equal. In the case of the beam with low concrete strength, since E value will be lower, to compensate for this e_c and k_{nd} values must be larger, which means the movement of neutral axis toward tension side. This causes a shortening in the internal moment arm jd shown in Figure 11. This situation can be better explained by referring to Figure 11. The following relationships exist between beams A and B on Figure 11.

$$f'c_A < f'c_B$$

$$Ec_A < Ec_B$$

$$es_A = es_B$$

$$Fs_A = Fs_B$$

$$Fc_A = Fc_B$$

$$ec_A > ec_B$$

so $d_A < d_B$ and $M_A < M_B$

where

f'c = cylinder strength of concrete

es = steel strain

F_s = tensile force at the cross section

E_c = Young's modulus of concrete

F_c = compressive force on concrete

e_c = strain of concrete at the top fibers

d = internal moment arm

M = moment at the cross section

Where the above relationships for two beams with the same kind of reinforcement but with different strengths of concrete show that at any given steel strain, the moment capacities of the beams differ noticeably.

It is also strongly believed that the formation of cracks affect the ultimate strength of the beam greatly. The following figure taken from a paper by Ngo and Scordelis (18) shows the distribution of stress in the steel of a reinforced concrete beam with cracks.

As it is seen from Figure 12 in section A-B of the beam where moment is constant, the steel stress is not constant and approximately 350 per cent greater at the location of a crack than at a place away from the crack. This means that failure will occur at cross section where the crack is, while the other portions of steel are at stress far lower than the yield stress. This proves that early formation of cracks decreases the ultimate strength of a beam greatly because a crack causes the steel to reach a much higher stress

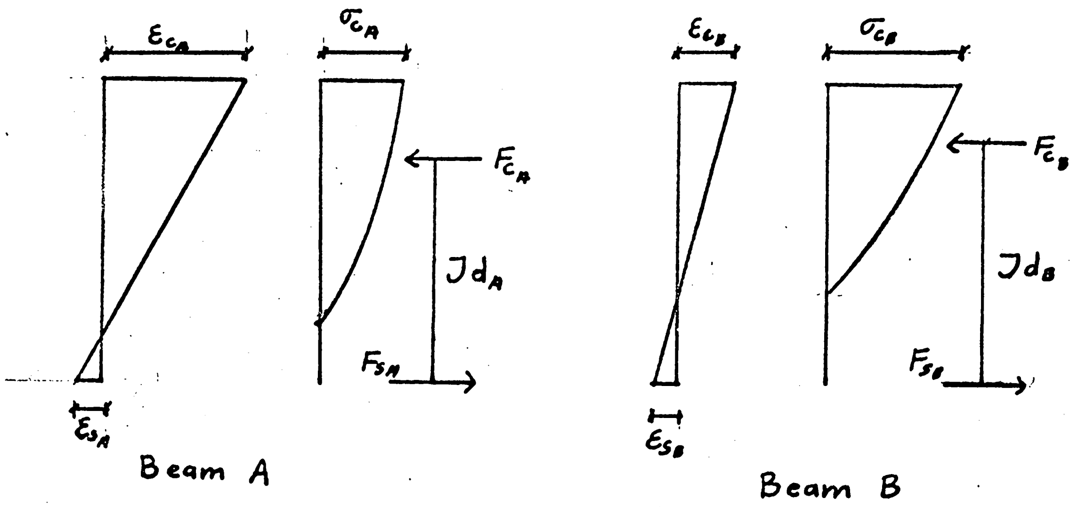

Figure 11. Stress and Strain Diagrams for Cross Sections of Beams A and B.

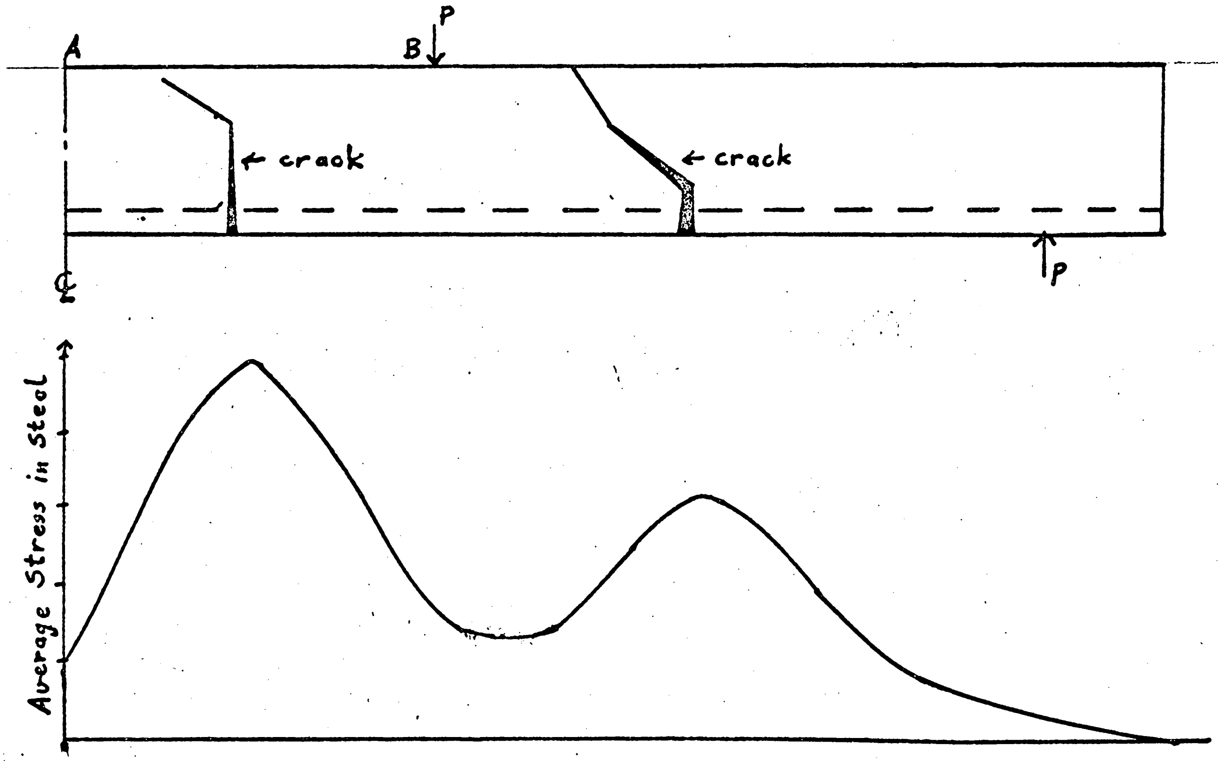

Figure 12. Stress Distribution of the Reinforcement of a Beam with Cracks.

than it will reach if there is not a crack at that location under the same loading conditions.

This fact must be certainly another factor which causes a decrease in the ultimate strength of the beams with C/S ratio equal to 0.20. Since the tensile stress of this mortar is much lower than the mortar with C/S ratio equal to 0.40, cracks in the beams with C/S ratio equal to 0.20 form at loads much lower than the loads which cause cracking in beams with C/S equal to 0.40. Early formation of cracks means steel reaching high stresses at early stages of loading which causes failure at much lower loads.

Also the same fact is justified by the comparison of the beams with initial cracks with the identical beams without initial cracks. Figures 19 and 20 show how the initial cracks affect the behavior of the beams.

Obviously the deflection of the beam follows the same pattern, that is deflections increase in the case of the beams with C/S equal to 0.20 more rapidly than in the case of the beams with C/S equal to 0.40. Figures 21 through 23 show this clearly.

C. Reinforcement Analysis

A direct comparison of the effects of the different kinds of reinforcing is quite difficult. Although the exact stress-strain relationship for #2 plain bars is known, not much is known about the behavior of expanded steel lath and

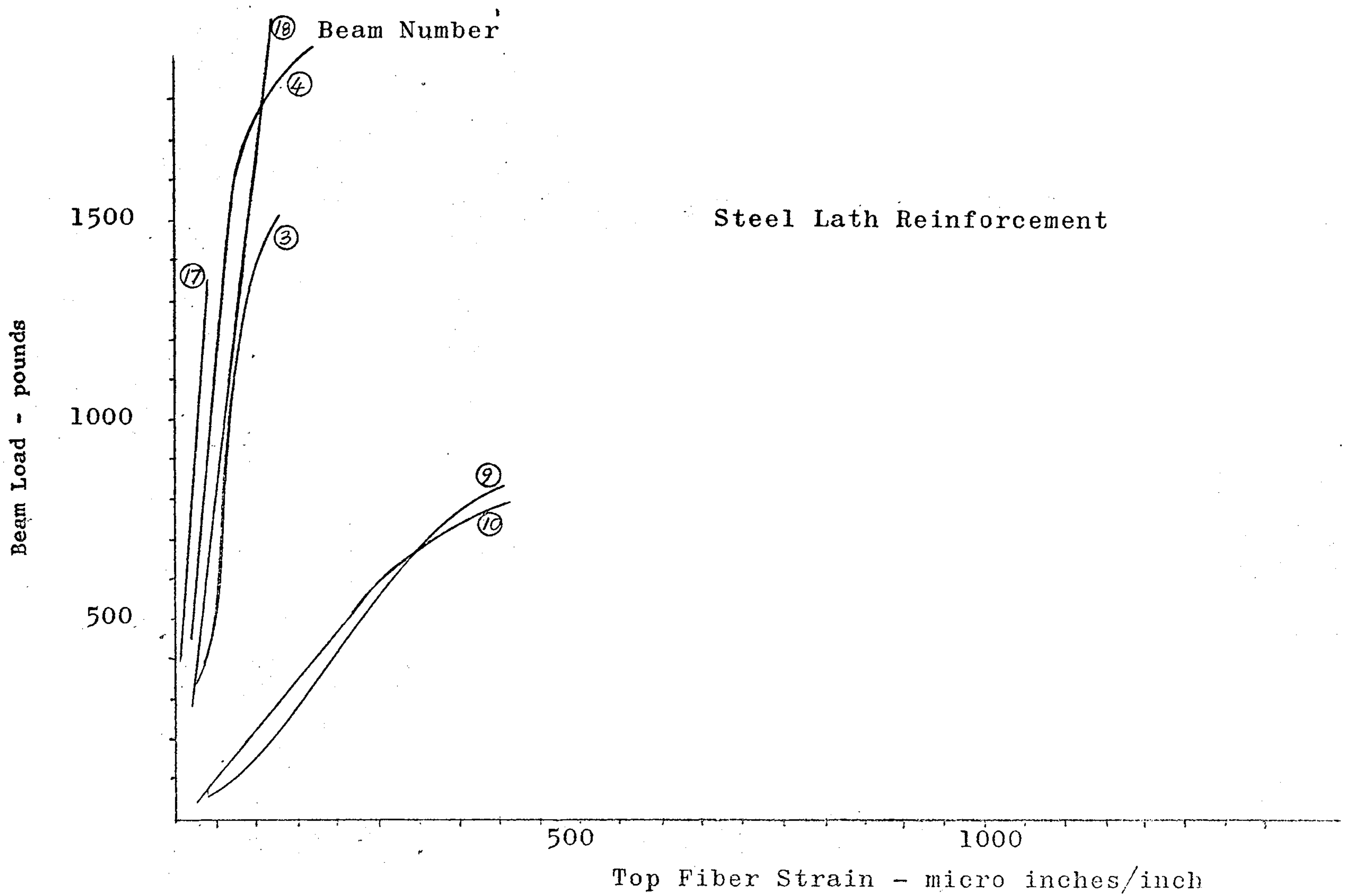

Figure 13. Load-Strain Curves.

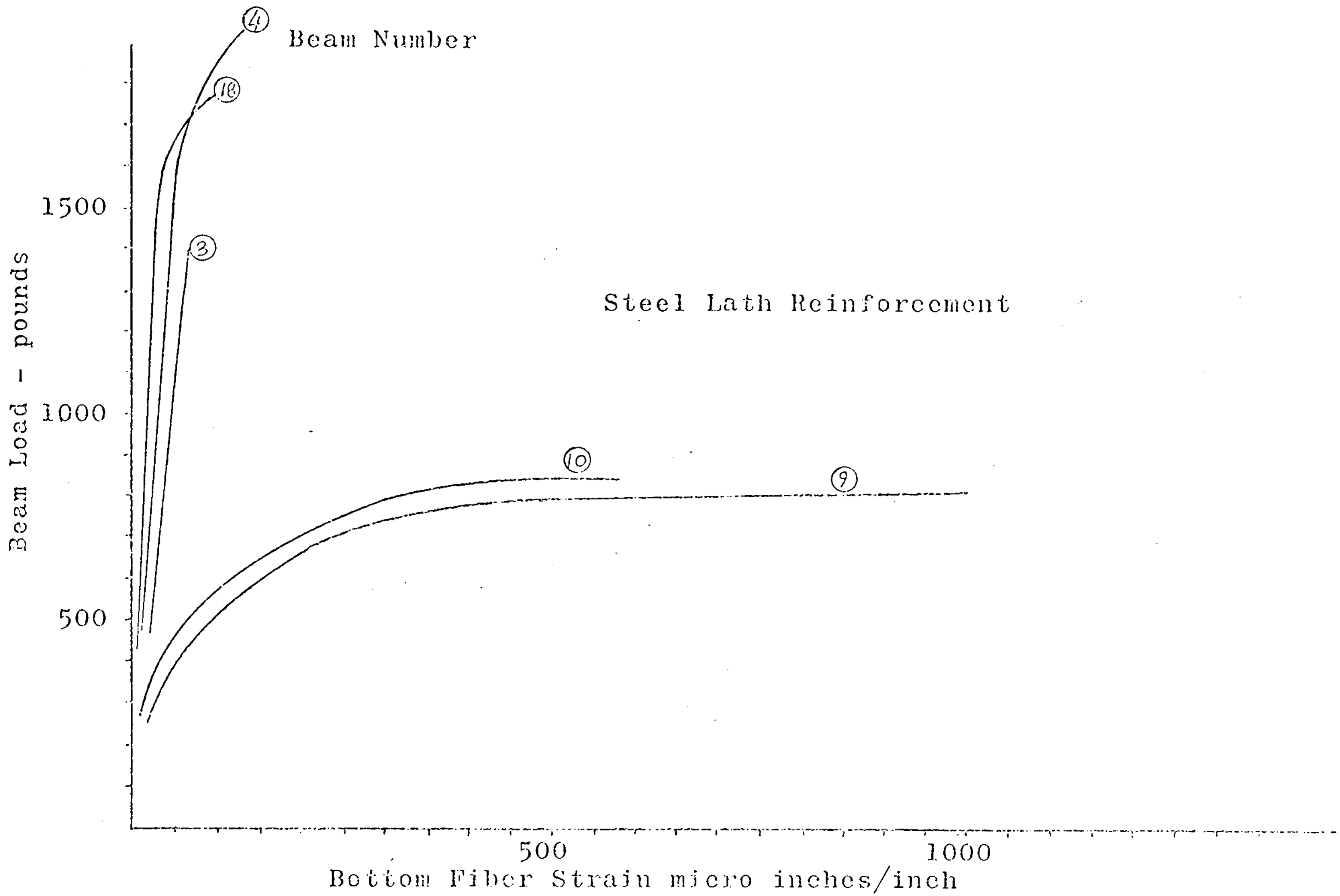

Figure 14. Load-Strain Curves.

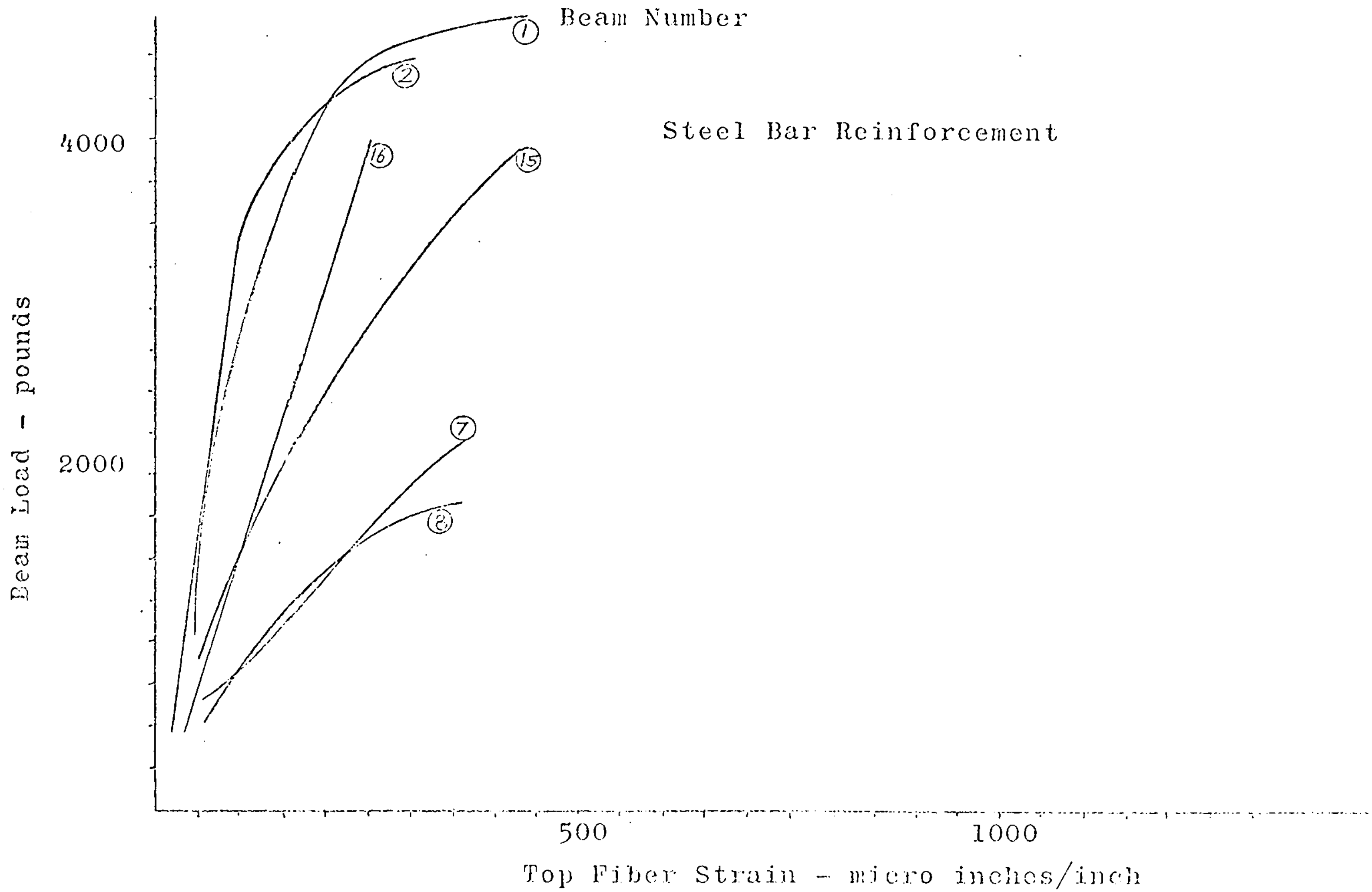

Figure 15. Load-Strain Curves.

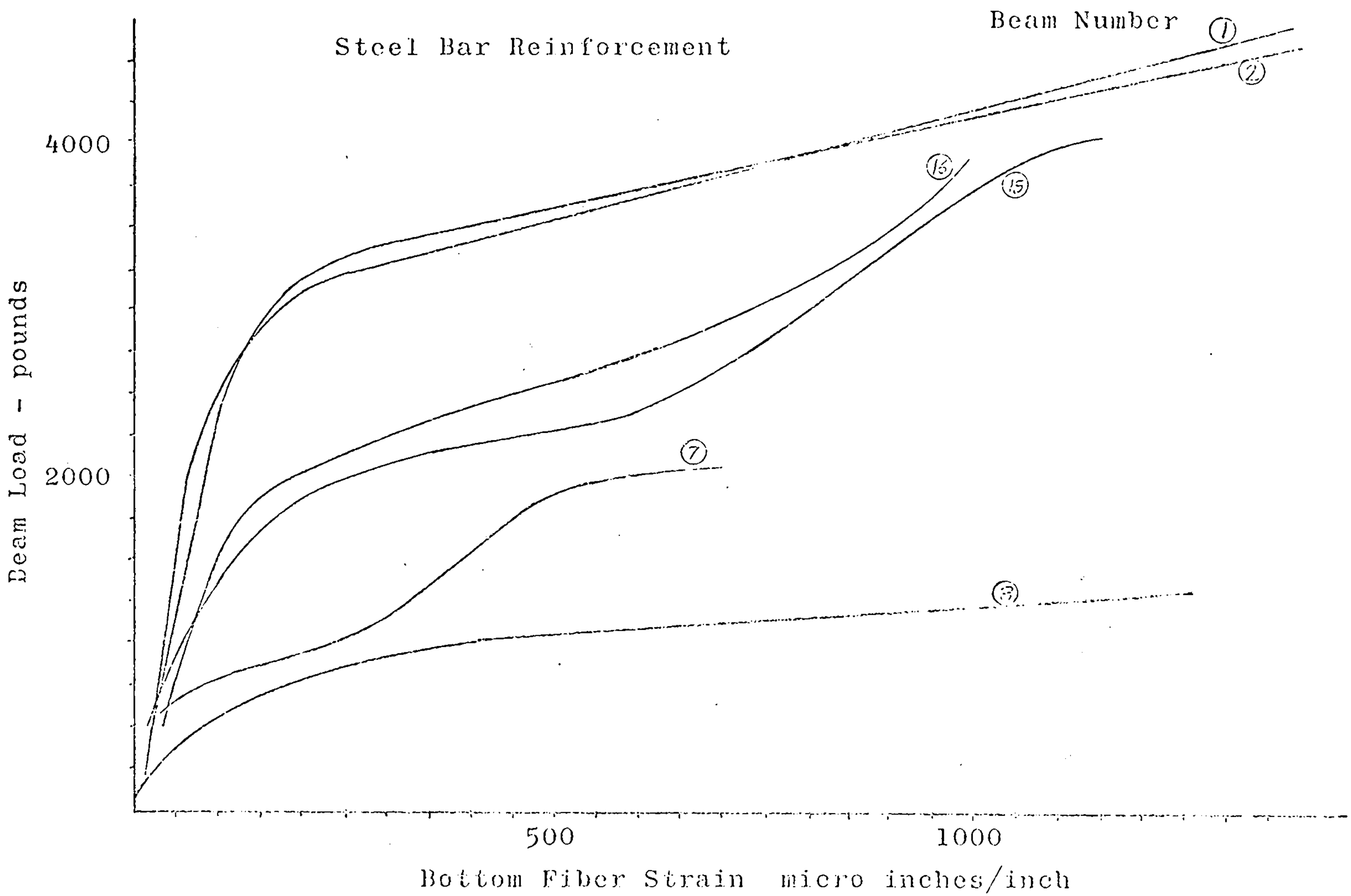

Figure 16. Load-Strain Curves.

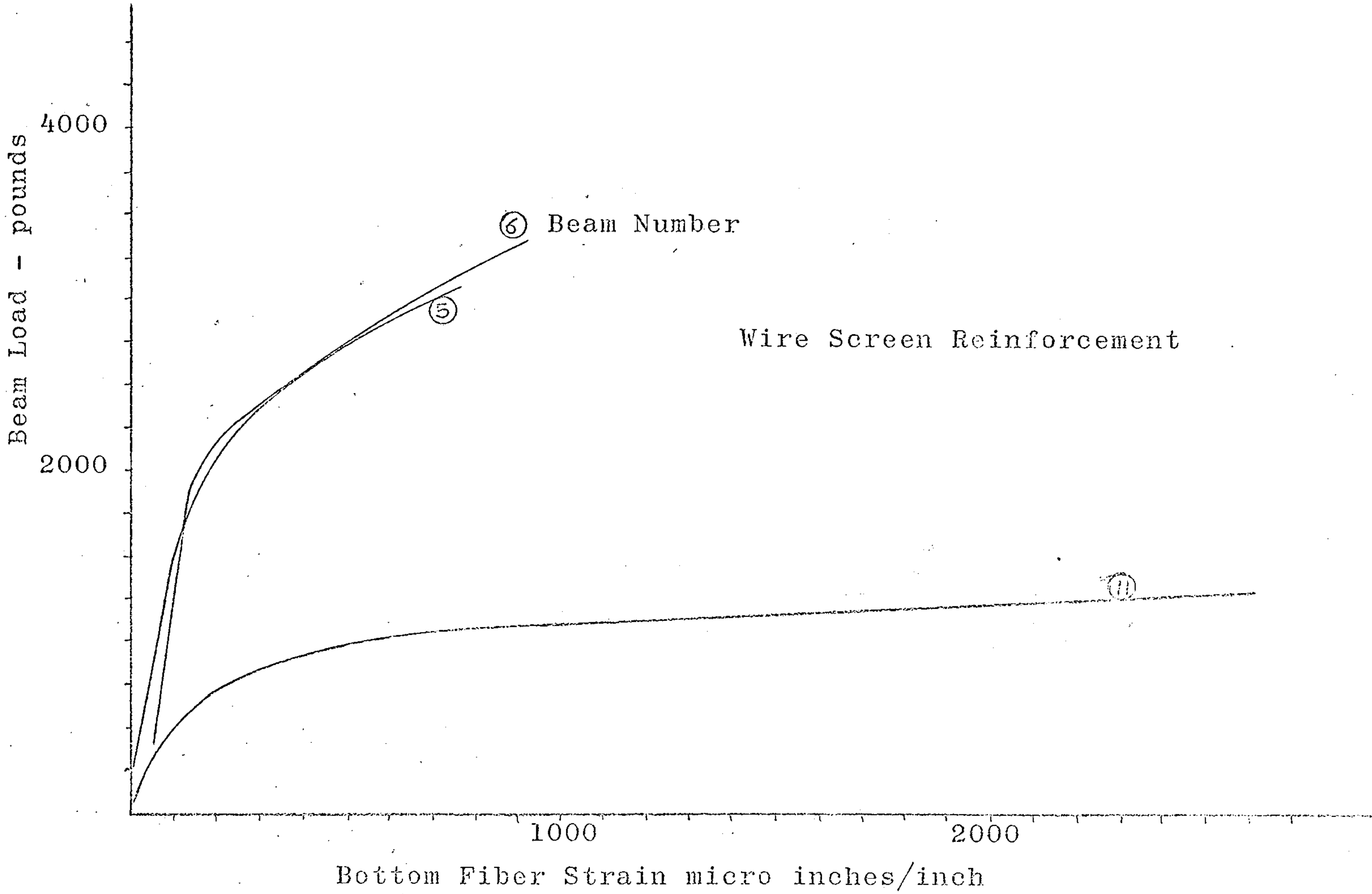

Figure 17. Load-Strain Curves.

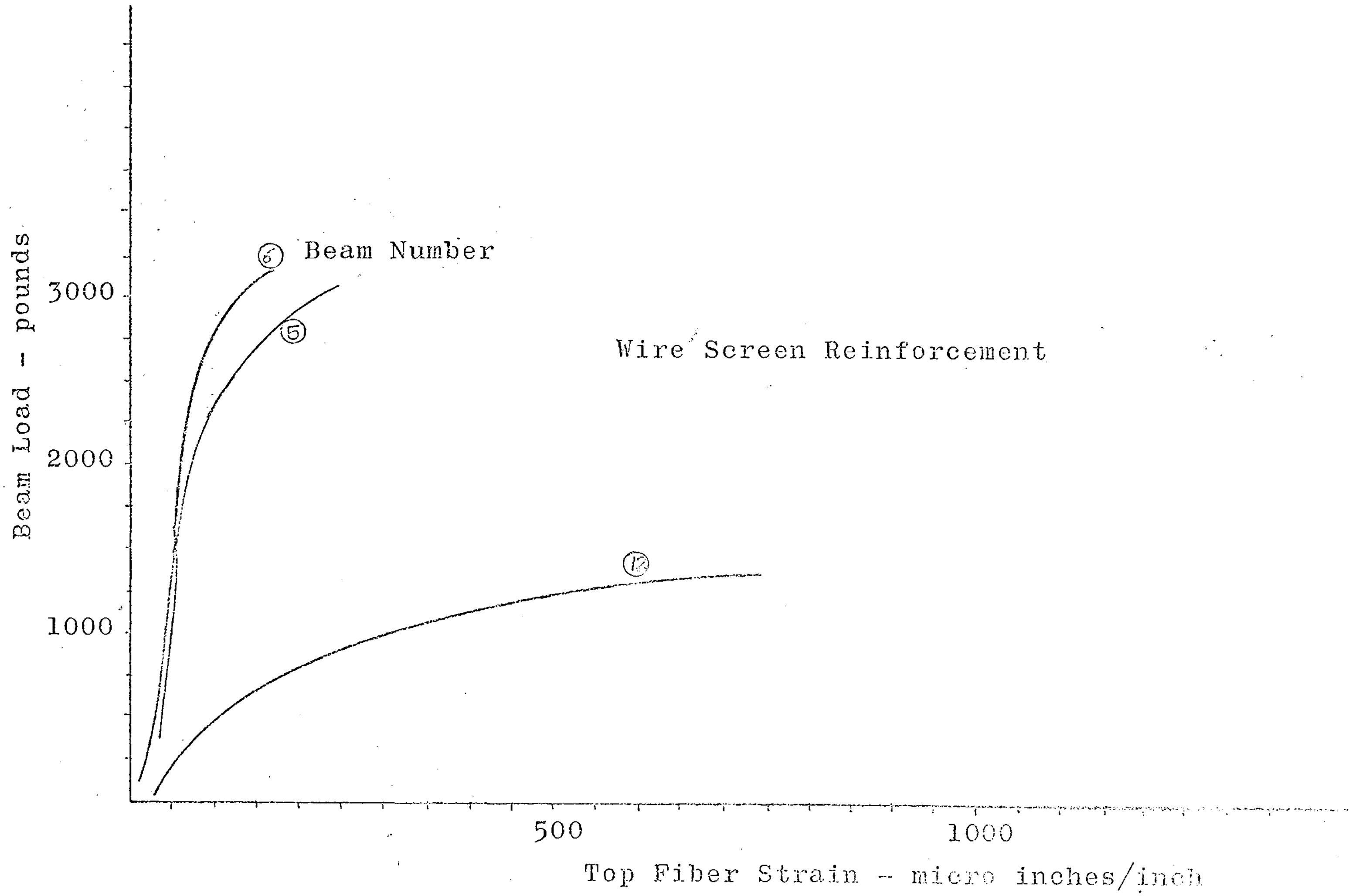

Figure 18. Load-Strain Curves.

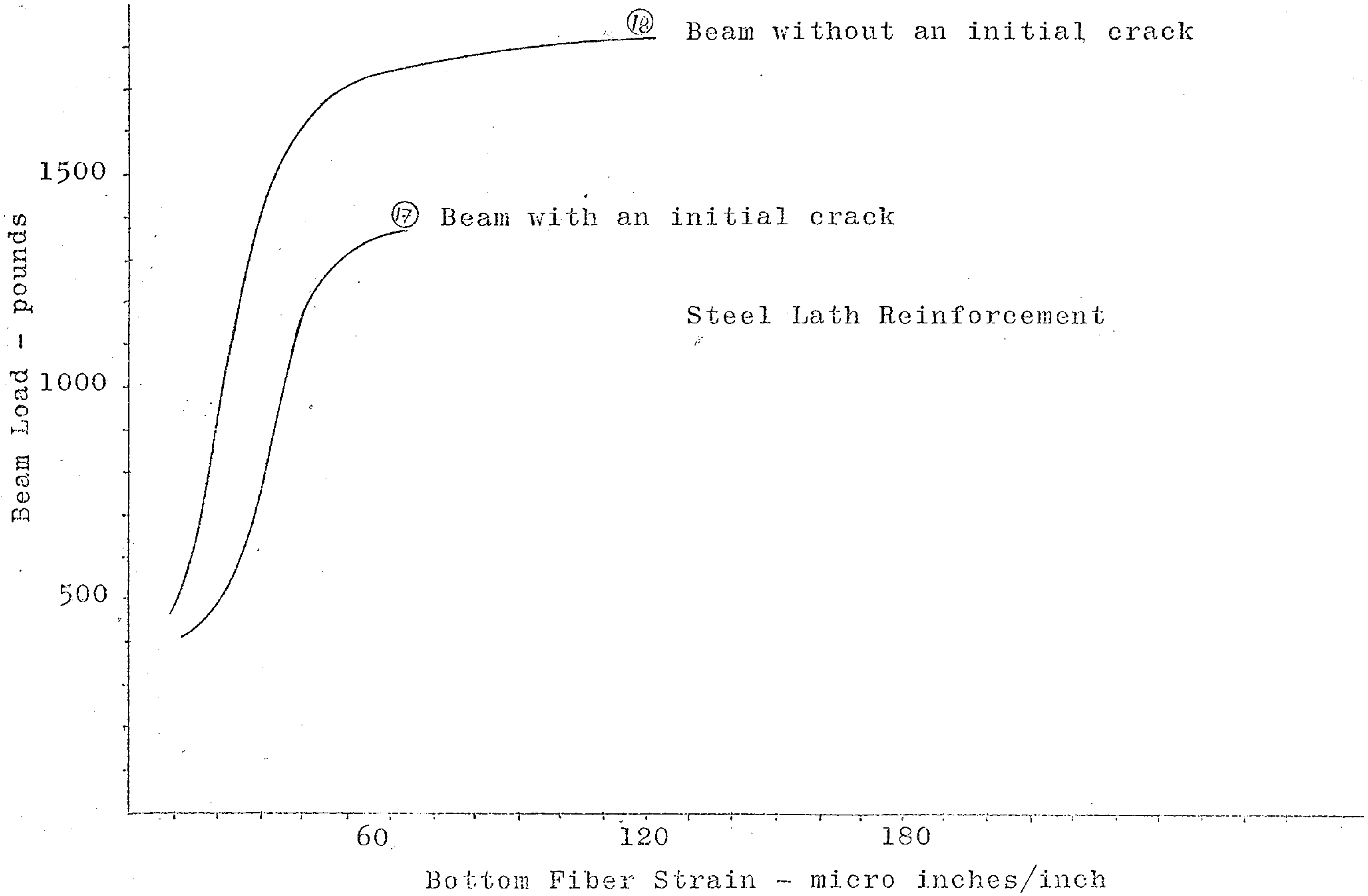

Figure 19. Load-Strain Curves.

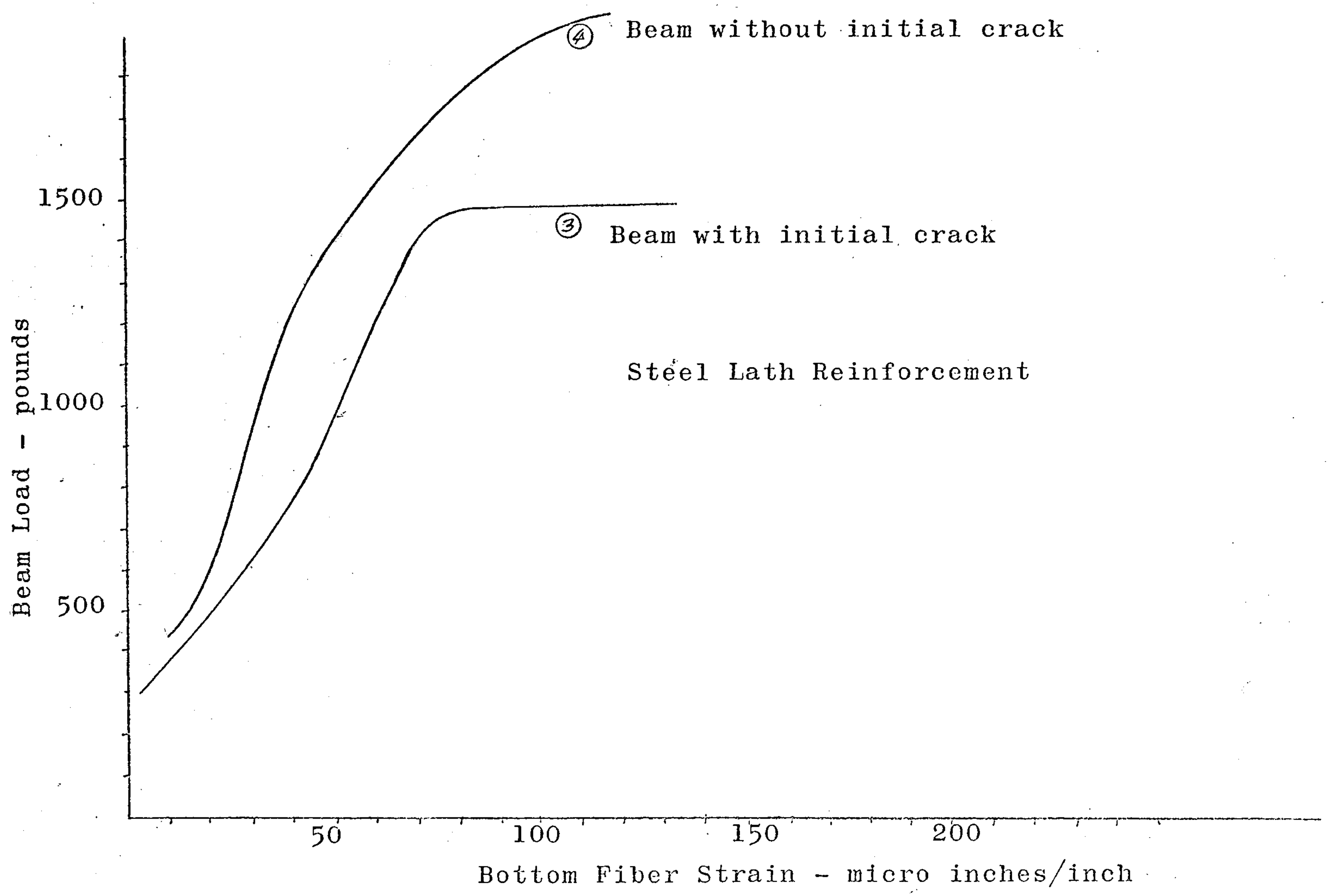

Figure 20. Load-Strain Curves.

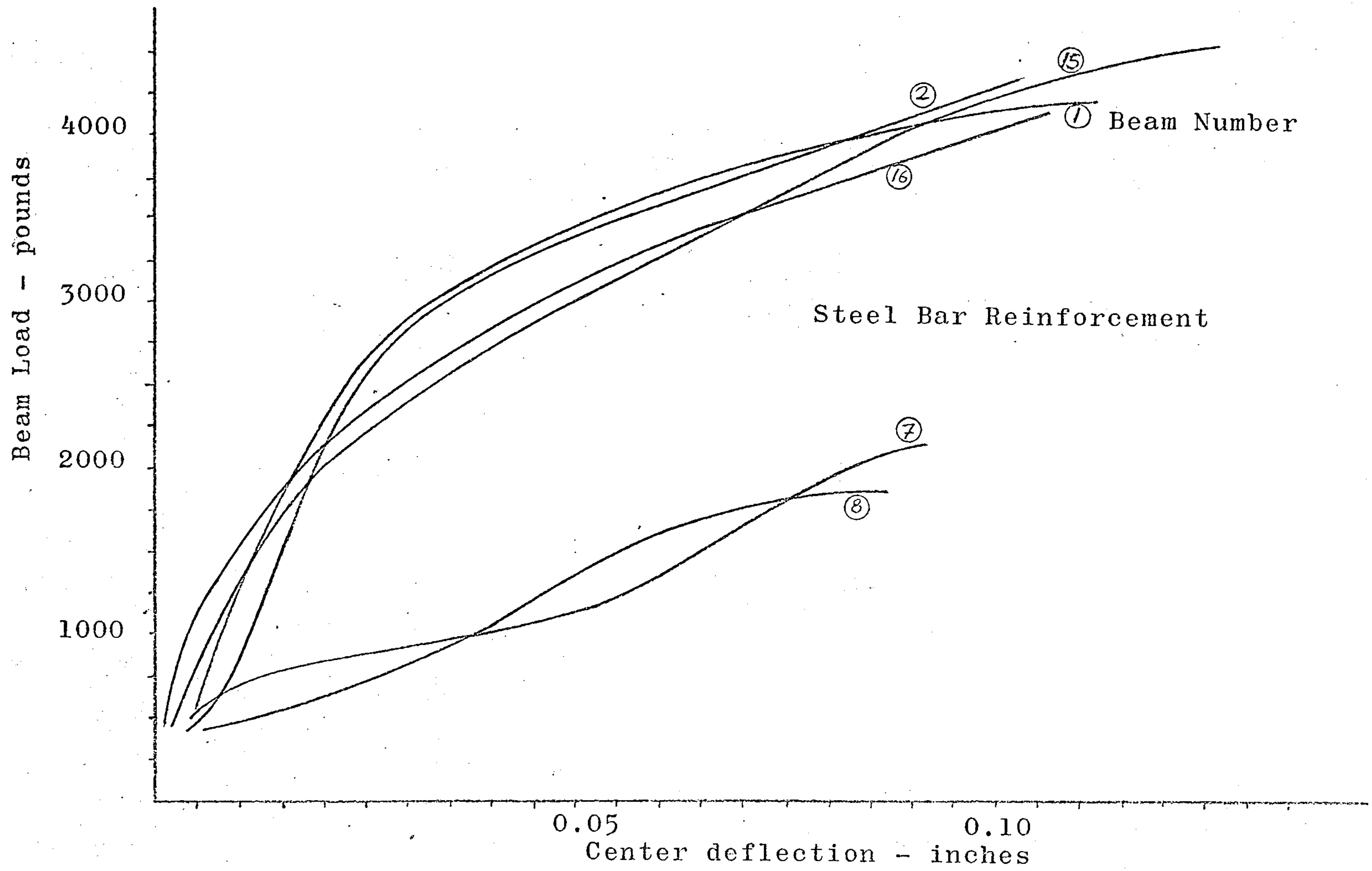

Figure 21. Load-Deflection Curves.

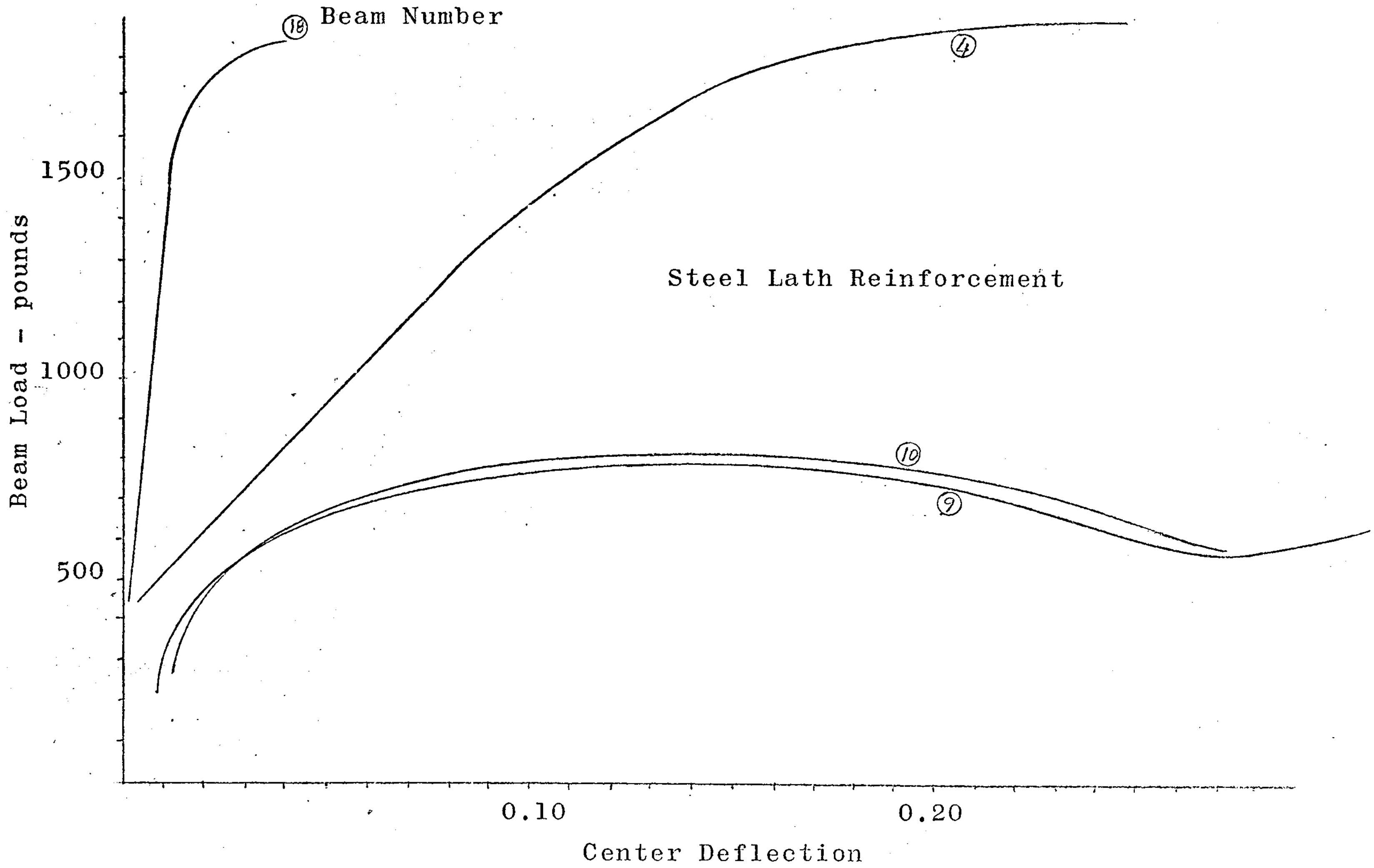

Figure 22. Load-Deflection Curves.

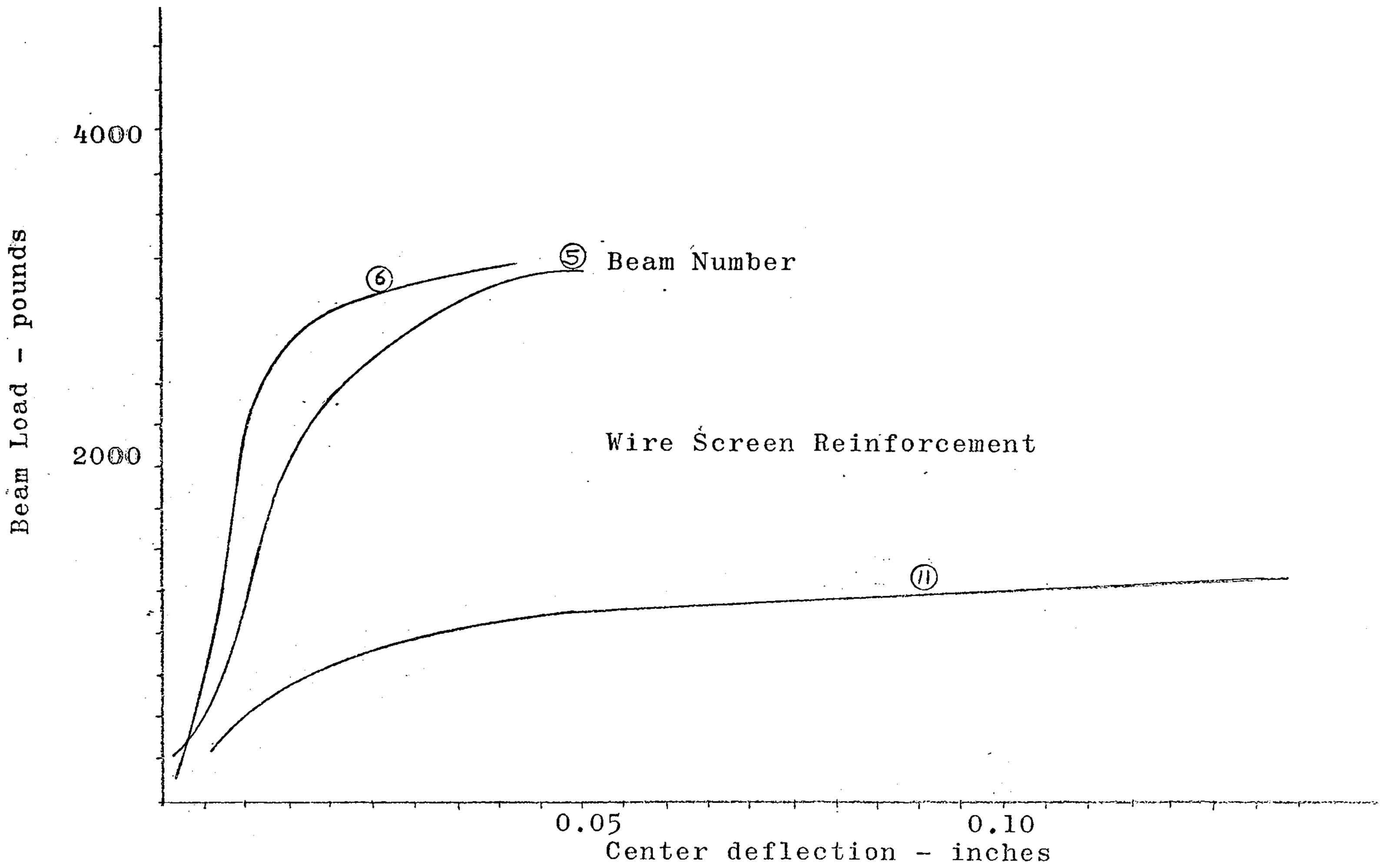

Figure 23. Load-Deflection Curves.

steel wire screen. Expanded steel lath does not normally fail by yielding because the threads break off prematurely at the notches produced during piercing of the steel sheet, so that full use of the steel strength can not be made. The steel threads in the wire screen appear to have the properties of cold drawn steel wire with approximately the same yield strength and a short yielding range.

Observations on the tested beams showed that except in those beams in which horizontal cracks formed along the reinforcement the expanded steel lath and the screen wire were not able to straighten because they were embedded in the mortar. The assumption can then be made that strains measured at the lower fibers of the beams are related to the stress by

$$\sigma = (\varepsilon)(E)$$

If it is also assumed that E is the same for all three kinds of steel reinforcement, the analysis can be based on the steel ratios. Values of steel areas give the following ratios:

$$\frac{\text{Area of expanded steel lath (Beams 3, 4, 9, 10)}}{\text{Area of \#2 steel bars (Beams 1, 2, 7, 8)}} = 0.51$$

$$\frac{\text{Area of steel wire screen (Beams 5, 6, 11, 12)}}{\text{Area of \#2 steel bars (Beams 1, 2, 7, 8)}} = 0.46$$

$$\frac{\text{Area of steel wire screen (Beams 5, 6, 11, 12)}}{\text{Area of expanded steel lath (Beams 3, 4, 9, 10)}} = 0.90$$

Ratios of average loads causing first cracks are as follows:

$$\frac{\text{Load causing first crack in Beams 3, 4}}{\text{Load causing first crack in Beams 1, 2}} = 0.50$$

$$\frac{\text{Load causing first crack in Beams 5, 6}}{\text{Load causing first crack in Beams 1, 2}} = 0.86$$

$$\frac{\text{Load causing first crack in Beams 5, 6}}{\text{Load causing first crack in Beams 3, 4}} = 1.75$$

$$\frac{\text{Load causing first crack in Beams 9, 10}}{\text{Load causing first crack in Beams 7, 8}} = 1.30$$

$$\frac{\text{Load causing first crack in Beams 11,12}}{\text{Load causing first crack in Beams 7, 8}} = 2.21$$

$$\frac{\text{Load causing first crack in Beams 11,12}}{\text{Load causing first crack in Beams 9, 10}} = 1.70$$

When the percentages of steel and the percentages of loads causing first cracks are compared it is seen that steel wire screen is very effective in delaying the formation of cracks. Compared with plain steel bars, it is seen that in the case of beams with C/S (cement-sand) ratio equal to 0.40 it is 1.88 times as effective in delaying cracks. In the case of beams with a C/S ratio equal to 0.20 this number goes up to 4.43.

Steel wire screen is also superior to expanded steel lath. In the case of beams with a C/S ratio equal to 0.40, it is seen that steel wire screen is 1.92 times as effective in delaying cracks than expanded steel lath. In the case of the beams with a C/S ratio equal to 0.20 this number is 1.90.

In the case of beams with a C/S ratio equal to 0.40 the effects of plain bars and expanded steel lath are almost the same but in the case of beams with a C/S ratio equal to 0.20, expanded steel lath is 1.3 times as effective as plain steel bars in delaying the formation of tension cracks.

An analysis of Figures 24 and 25 shows that besides the amount of reinforcing steel, the type of reinforcing steel affects the load carrying capacities of the beams.

Comparing the beams reinforced with plain steel bars and steel wire screen, it is seen that at lower fiber strains (up to about 100 micro-inch inches) the ratio of load capacities is almost equal to the ratio of steel areas. But as the lower fiber strain increases it is seen that the beams with steel wire screen carry more load so that the ratio of load capacities at a given fiber strain level is greater than the steel area ratio and approaches about 0.80.

If beams with expanded steel lath and plain steel bars are compared it is seen that in the case of beams with a C/S ratio equal to 0.40 the ratio of load capacities at a given lower fiber strain is the same as the ratio of steel areas. In the case of beams with a C/S ratio equal to 0.20, in the strain range of 200 to 400 micro-inch inches, the ratio of load capacities becomes larger than the ratio of steel areas.

This behavior of the beams can be related directly to the formation of cracks. At the points where the cracks occur the steel obtains higher stresses than at the other portions of the steel. As cracks form and increase in number the beam gets weaker.

In cases where the formation of cracks are delayed, load carrying capacity stays higher than in cases where the cracks form early.

D. Thin Beam Analysis.

The behavior of these beams confirmed the conclusions drawn from the tests of other beams; as was expected, due to its thin cross section the bending deflection was excessive. Since the expanded steel lath used in this beam as reinforcement was not capable of straining much, the lowest layer of expanded lath yielded before the other layers were strained greatly so that the fluctuation of load carrying capacity occurred as shown in Figure 26.

E. Analysis of Ferro-Cement.

There are three main factors that make ferro-cement less economical than normal reinforced concrete. These are:

1. More labor hours to place concrete.
2. Use of higher percentages of cement.
3. Use of more expensive kind of steel as reinforcement.

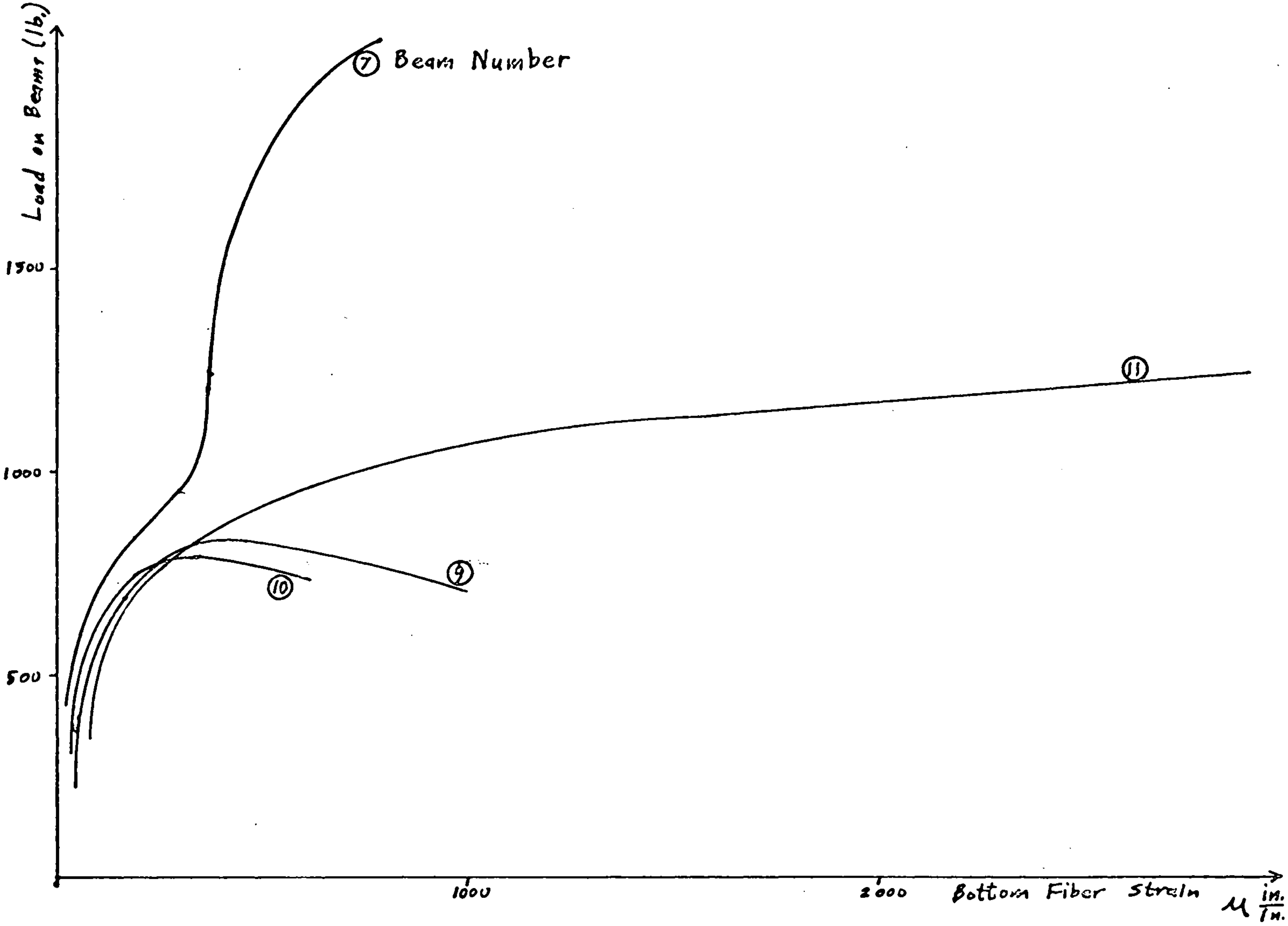

Figure 24. Load-Strain Curves.

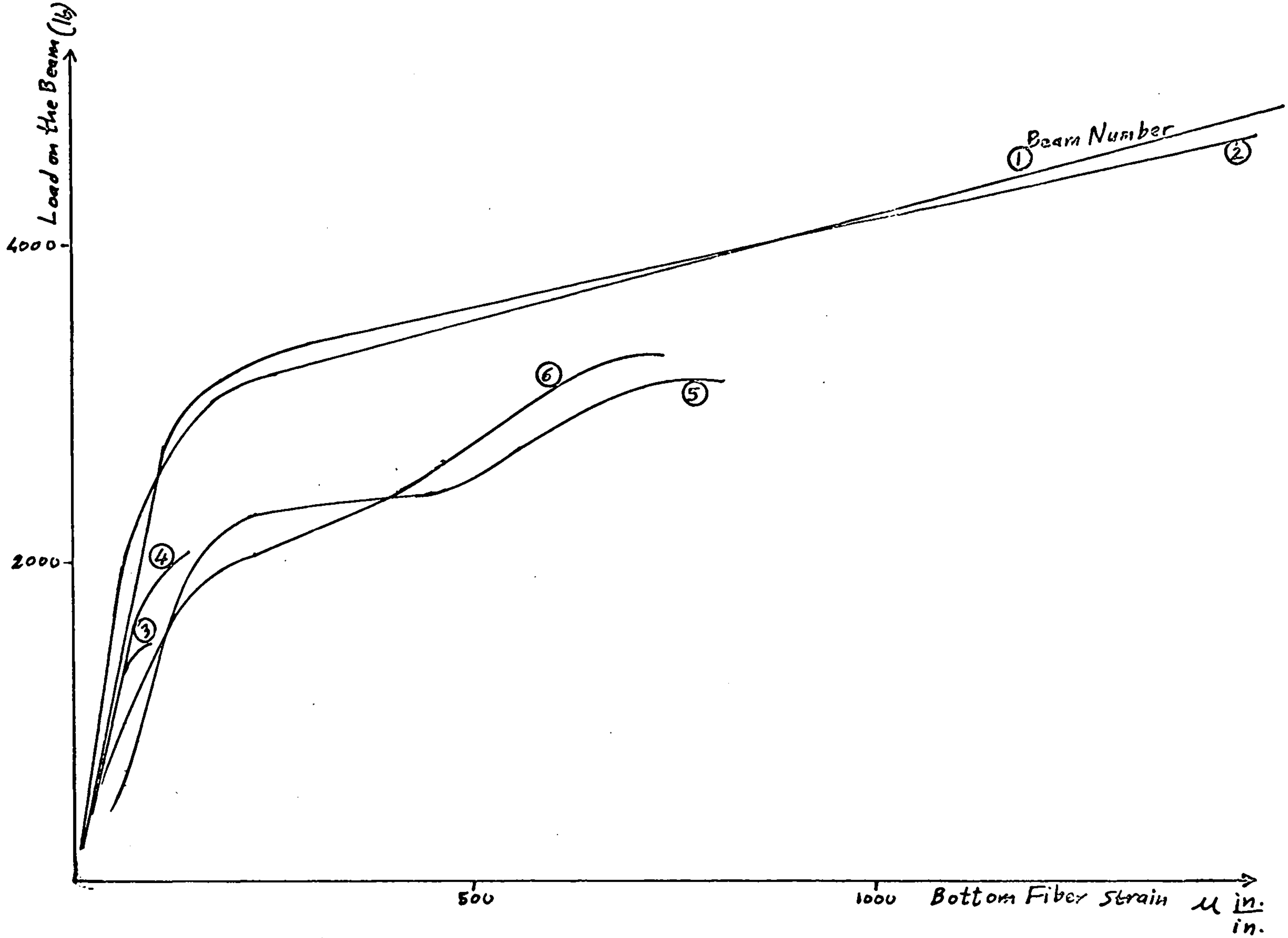

Figure 25. Load-Strain Curves.

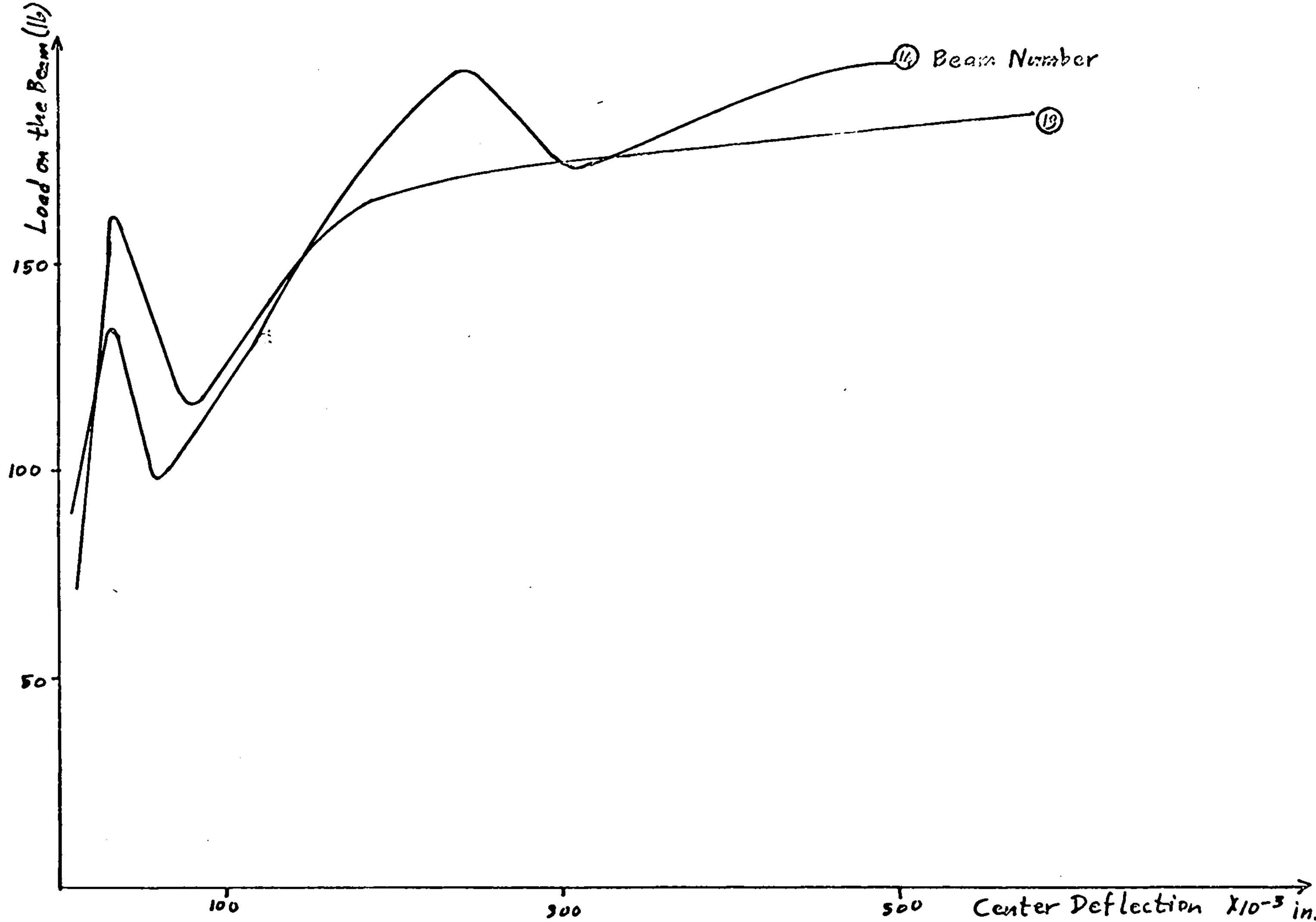

Figure 26. Load-Deflection Curves.

Since the reinforcement is placed in layers with little space between them the mortar has to be placed by hand and must be compacted by hammering. This procedure requires more man hours than normal concrete.

In the production of the test beams it took 80 per cent more time to prepare those with expanded steel lath reinforcement and 120 per cent more time to prepare those with wire screen reinforcement than to prepare the normal reinforced concrete beams.

In normal concrete the percentage of cement varies from 12 per cent to 18 per cent depending on the amount of fine aggregates used. In ferro-cement up to 70 per cent has been used. In the tests here reported 20 per cent cement did not produce the desirable properties of ferro-cement but 40 per cent gave results. Considering 40 per cent as the normal percentage of cement in ferro-cement, and 17 per cent as the average percentage of cement in normal concrete, it appears that ferro-cement requires about two and one-half times as much cement as normal concrete.

Expanded steel lath, steel wire screen or other similar types of steel reinforcement is likely to be more expensive than normal steel reinforcement. The costs of reinforcement used in this project are as follows:

1. One foot of reinforcing bar with cross sectional area equal to 0.05 in.^{2} costs six cents.

2. One foot of galvanized wire screen with effective cross sectional area equal to 0.05 $in.^2$ costs 18 cents.
3. One foot of expanded steel lath with effective cross sectional area equal to 0.05 $in.^2$ costs 6.9 cents.

The extra number of man hours required to place the mortar can be reduced by using new techniques such as concrete shooting or pumping the mortar in between the layers of reinforcement. Especially in cases of prefabricated construction, these techniques may be modified and used successfully.

The cost increase due to the use of high percentages of cement may not be as high as it seems at the first glance. As shown in the analysis Section D, use of a high percentage of cement works very efficiently to increase the load capacity of beams. This means that smaller sized beams can be used with less dead weight which means less mortar and less cement. Also in deep beams a combination of mortar around the reinforcement and normal concrete above can be used very successfully as shown by beams 17 and 18. This decreases considerably the amount of cement used.

The reinforcement used in ferro-cement are the steel products produced for other purposes so that they may not be extremely effective. For example, expanded steel lath which is not much more expensive than steel bars can not reach the stresses reached by steels due to the notches produced during manufacturing. Galvanized steel wire screen which works very well in ferro-cement, contains equal amounts of threads in both directions so it is equally strong in two directions while it is only used in one direction in the beams. It can be suspected that the transverse wires of steel are wasted.

In constructions of structures such as plates and shells, where reinforcement in two directions is required, screen type reinforcement may prove to be very economical.

If ferro-cement becomes popular, special screen type reinforcement can probably be produced more economically for this purpose.

V. DISCUSSION

Until now ferro-cement has only been considered for use in the construction of structural elements with thin cross sections such as plates and shells, pressure pipes and concrete boats. In all these cases it has proved to be good due to its capacity to develop relatively high strength and water tightness with thin cross sections.

Although prestressed concrete provides the same advantages, in some cases ferro-cement may prove to be more economical, especially when irregular shapes of small prefabricated elements are used. Also in some complicated structures where prestressing is not possible to use, ferro-cement may be the best solution.

Although there is no sign of it being used in deep beam elements, the idea of ferro-cement can be very well applied with some revisions in design and construction. The tests performed for this thesis show that beams reinforced in the manner of ferro-cement, that is, thin reinforcing elements placed in layers closely spaced with mortar in between, proved in some cases to be superior to normal reinforced concrete beams. If some practical solution can be found to overcome the technical difficulties of placing this kind of reinforcement, it can be widely used in deep members.

VI. CONCLUSION

Basically the effect of two parameters were investigated in this thesis. They were the percentage of cement and the kind of steel reinforcement that were typical of those used in the manufacture of ferro-cement. A comparison of ferro-cement and normal reinforced concrete beams were also made.

Of two different mixes of mortar the one with a cement-sand ratio equal to 0.40 showed some favorable properties of high tensile and compressive strengths and high resistance to cracking.

Of three kinds of reinforcing steel used, galvanized steel wire screen was superior to plain steel bars and expanded steel lath in delaying the formation of cracks and developing higher beam capacities.

Expanded steel lath is less expensive than galvanized steel wire screen but did not prove to be as good because of premature breaking at the notches at lower stresses.

The ability of ferro-cement to develop high strength as suggested by Nervi was found to be true. This was attributed to the fact that the mortar and the reinforcing steel were quite effective in delaying the formation of cracks and preventing premature loss of strength.

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For the assistance which I received during the experimental phase I wish to thank the laboratory technicians,

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AN INVESTIGATION OF FERRO-CEMENT

by

Ugur Dincer

Abstract

In this study the properties of ferro-cement are investigated. Ferro-cement is a kind of reinforced concrete differing from normal reinforced concrete in that cement mortar and fine steel reinforcement are used rather than aggregate concrete and bar reinforcement.

Concrete consistency and three kinds of reinforcement are taken as the parameters which affect the properties of the ferro-cement. Tests are carried out on beams subjected to bending while varying these parameters. A comparative analysis of the experimental results is made and some analytical explanations are included.

It was found that mortar with high cement content and fine mesh reinforcement delayed the crack formations and increased the ultimate load carrying capacities of the beams.