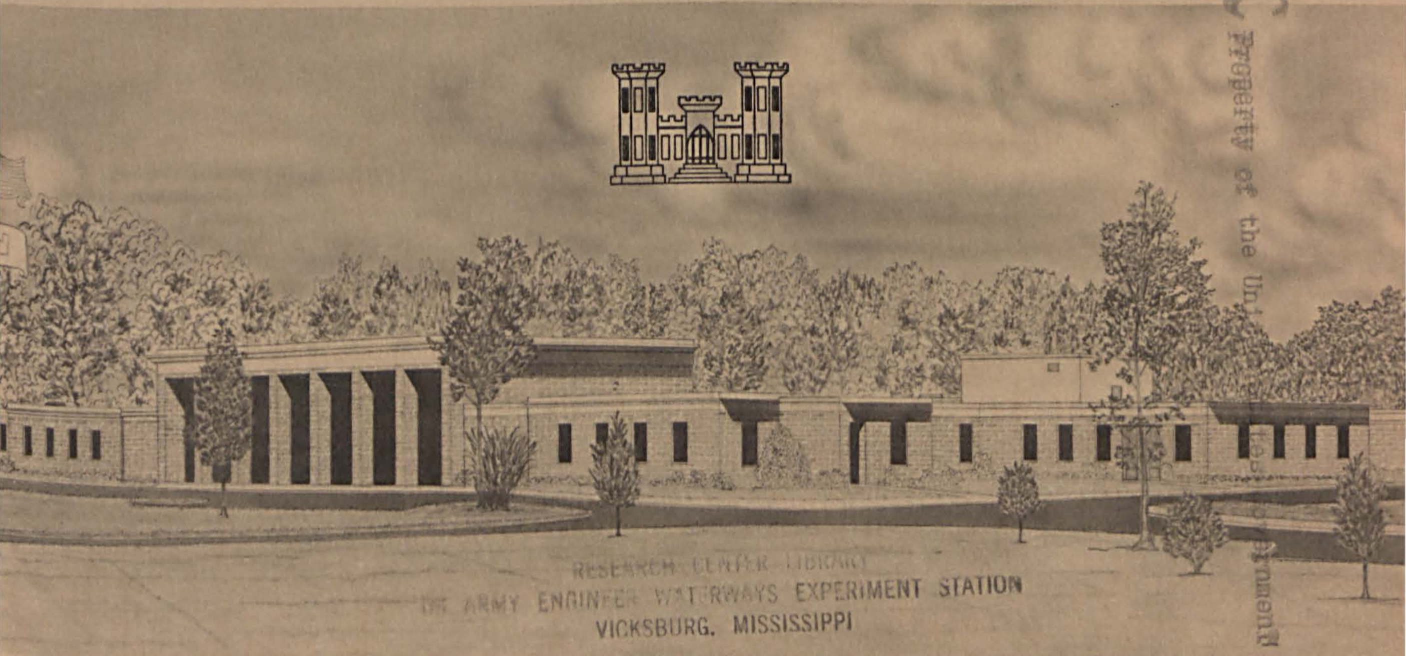
MISCELLANEOUS PAPER N-71-2

# SIMILITUDE STUDY OF REINFORCED CONCRETE DEEP BEAMS

by

J. P. Balsara, L. E. Roggenkamp







January 1971

Sponsored by Defense Atomic Support Agency

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# ABSTRACT

Tests were performed on deep beams under a midspan load to determine the scaling of cracking and ultimate load-carrying capacities of beams failing in shear. Two types of scaling procedures were used, one in which only the geometries are scaled (replica or mach models), the other in which both geometry and material properties are scaled (dissimilar-strength or environmental models). The results of twenty simply supported beams tested statically with span-to-depth ratios of 4.67, 3.88, 2.80, and 2.00 and comprising 1/4- and 1/2-scale models and laboratory prototypes are presented. Two prototype beams with L/d ratios of 4.67 and 3.88 were tested dynamically to provide some correlation between statically and dynamically loaded beams. Test results indicate that cracking loads can be adequately predicted from both replica and dissimilar-strength models and ultimate loads can be predicted from replica models for all span-to-depth ratios tested. When transition from beam to arch action occurs, the dissimilar-strength models underpredict the ultimate load-carrying capacity of the prototypes.

## PREFACE

The research reported herein was conducted at the U. S. Army Engineer Waterways Experiment Station under the sponsorship of the Defense Atomic Support Agency. The results of this study were presented at the American Society of Civil Engineers National Structural Engineering Meeting held at Portland, Oregon, 6-10 April 1970. The work was accomplished during the period 1968-1970 under the general supervision of Mr. G. L. Arbuthnot, Jr., Chief of the Nuclear Weapons Effects Division, and under the direct supervision of Mr. W. J. Flathau, Chief of the Protective Structures Branch. The report was prepared by Dr. J. P. Balsara and Mr. L. E. Roggenkamp.

COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE, were the Directors of the Waterways Experiment Station during this study and the preparation of this report. Mr. J. B. Tiffany and Mr. F. R. Brown were Technical Directors.

CONTENTS

## NOTATION

| | |
|---|---|
| $a$ | Length of the shear span |
| $b$ | Width of beam cross section |
| $d$ | Effective depth, distance from top compressive fiber to centroid of tensile reinforcement |
| $f'_c$ | Concrete compressive strength |
| $F$ | Force dimension |
| $h$ | Total depth of beam |
| $K_{f'_c}$ | Scale factor for concrete strength |
| $K_L$ | Scale factor for length |
| $L$ | Length dimension; clear span, distance between inside edges of supports |
| $M$ | Bending moment |
| $p$ | Ratio of area of tensile reinforcement to effective area of concrete |
| $p'$ | Ratio of area of compressive reinforcement to effective area of concrete |
| $P_u$ | Ultimate load |
| $V$ | Shear force |
| $V_c$ | Cracking shear |
| $V_u$ | Ultimate shear |

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows.

| Multiply | By | To Obtain |
|---|---|---|
| inches | 25.4 | millimeters |
| kips | 4.448222 | kilonewtons |
| pounds per square inch | 6.894757 | kilonewtons per square meter |

# SIMILITUDE STUDY OF REINFORCED CONCRETE DEEP BEAMS

## INTRODUCTION

The design of protective structures to resist high-intensity loading from nuclear detonations must often be experimentally evaluated, and in many instances prototype testing would exceed the load and/or size limitations of laboratory simulation facilities. This evaluation can be carried out by testing models and applying the principles of similitude.

The objectives of the similitude study are to develop design, construction, and testing techniques for scale modeling deep reinforced concrete structures.

As an initial step towards achieving these objectives, deep beams have been statically tested under a midspan load to determine the scaling of cracking and ultimate load-carrying capacities of beams failing in shear. Two types of scaling procedures have been used, one in which only the geometries are scaled (replica or mach models), the other in which both geometry and material properties are scaled (dissimilar-strength or environmental models).

The use of any modeling technique requires that the design of the model be based on parameters which influence the behavior of its prototype. Reference 1 presents an expression for shear at diagonal tension cracking which is derived from the equation of principal stress at a point. The equation contains two parameters which, when simplified, may be expressed as a nondimensional parameter $V/(bd\sqrt{f'_c})$ which represents the diagonal tension strength, and $(\sqrt{f'_c}/p)(M/Vd)$, a parameter with dimensions of $FL^{-2}$, which represents the properties of the beam. This parameter has dimensions since the modulus of elasticity of steel is assumed a constant and not included. The terms used in these expressions and their dimensions are defined as follows:

$V$ = shear force, $F$

$b$ = width of beam cross section, $L$

$d$ = effective depth of beam, $L$

$f'_c$ = concrete compressive strength, $FL^{-2}$

p = tensile reinforcement ratio

M = bending moment, FL

F = force dimension

L = length dimension

For simply supported beams with a concentrated load, the M/Vd term is synonymous with a/d, the ratio of shear span to effective depth.

Four series of static tests, A, B, C, and D, were conducted with the following respective span-to-depth ratios: 4.67, 3.88, 2.80, and 2.00. Model and prototype beams for each span-to-depth ratio were designed such that the quantity $\sqrt{f'_c}/p$ would be constant and that failure would occur in shear. As indicated by the beam designations in Table 1, each series included environmental (E), mach (M), and prototype (P) beams. Scale factors of 4 and 2 were used. Duplication of tests was used to indicate repeatability of results.

Two prototype beams were tested dynamically to provide some correlation between statically and dynamically loaded beams.

## EXPERIMENTAL PROCEDURE

The dimensions of the prototype beams are shown in Figure 1 and the properties of the beams tested statically are given in Table 1. All dimensions shown in Figure 1 were scaled by factors of 4 and 2 for the models. The area of the compressive reinforcement, in all cases, was half the area of the tensile reinforcement. Numbers 2, 4, and 8 intermediate-grade deformed bars were used for the 1/4-scale, 1/2-scale, and prototype beams, respectively, and anchor plates were provided for the tension steel.

A nominal concrete strength of 4,000 psi[1] was selected for the prototype beams and the 1/4-scale replica model, and 1,000- and 2,000-psi concretes were used for the 1/4- and 1/2-scale dissimilar-strength models. The maximum size aggregate (MSA) for the prototype beam was 1-1/2 inches, and scaled MSA's (i.e. 3/4 and 3/8 inch, respectively) were used for the 1/2- and 1/4-scale models.

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[1] A table of factors for converting British units of measurement to metric units is presented on page 8.

Two hydraulic loaders with capacities of approximately 25 kips and 500 kips were used for the static tests. These tests were conducted under a continuously applied load. Testing times varied from approximately 1 to 3 minutes. A 200-kip loader was used for the dynamic tests. Rapid loading in this loader is obtained by the use of rupture disks and the expansion of precompressed low bulk modulus fluid. Rise times are controlled by an orifice plate and by the stiffness of the beams. A more complete description of the loaders is presented in Reference 2.

All beams were instrumented to measure steel strains and deflections at midspan. Strain gages with scaled gage lengths of 1/4, 1/2, and 1 inch were bonded to the compression steel and both layers of tension steel. A few selected beams were gaged at quarterspan on the tension steel to determine strain distribution. Deflections were monitored by linear variable differential transformers (LVDT) at the top of the beam. The probe of the LVDT rested on the loading block and the body of the LVDT was anchored to the reaction frame. Loads were monitored by a load cell mounted at the end of the loading ram. Magnetic tapes or oscillograph recorders were used to record the data.

## RESULTS AND DISCUSSION

Standard compression cylinder tests were conducted periodically and the beams were tested when the strength of concrete was within approximately 10 percent of the desired value. Typical stress-strain curves from cylinder compression tests for the prototypes and models are shown in Figure 2a. The types of models are identified by a scale factor for length $K_L$ and a scale factor for concrete strength $K_{f'_c}$ . Figure 2b shows that the tensile strength of concrete can be approximated as a function of $\sqrt{f'_c}$ as assumed in the parameters for inclined cracking shear.

The normalized cracking shear $V_c$ and ultimate shear $V_u$ for the four series of models and prototypes are given in Table 2. The cracking load was interpreted from load-steel strain or load-deflection plots. Figures 3 and 4 show the normalized cracking and ultimate shear for the prototypes and models. The cracking shear parameter $V_c/(bd\sqrt{f'_c})$ is nearly constant since the parameter $(\sqrt{f'_c}/p)(a/d)$ representing the beam properties is approximately the same for all four span-to-depth ratios.

The ultimate shear $V_u/bd\sqrt{f'_c}$ shows an increase with decreasing L/d. Failures of beams with large span-to-depth ratio occur almost simultaneously with the development of the inclined crack. As the span-to-depth ratio increases, the beam can support loads considerably higher than the inclined cracking load due to a transition from beam to arch action. The area of concrete in compression above the crack and the tensile reinforcement, which acts as a tension tie, provide the increased load capacity. The magnitudes of the reserve capacity above cracking for the replica model and prototype beams with L/d ratios of 2.88 and 2.00 were similar to the values determined in Reference 3 for uniformly loaded beams and Reference 4 for beams loaded at third points. Reference 5 indicates that beams subjected to concentrated loads had lower reserve capacity than those subjected to uniform loads. The test results (Figure 3) show that increases in reserve capacity do occur but at smaller L/d ratios than observed in tests of uniformly loaded beams (the smallest value of L/d for those beams being 4.69).

Figure 5 shows similar crack development in the 1/4-scale replica model and prototype and the development of uniform strain in the tension steel after the formation of arch action. The similarity of crack patterns and failures of the 1/4-scale replica model and a prototype is shown in Figure 6. (No data were obtained for the prototype shown.)

Figures 7 through 10 show the posttest crack patterns for 1/4-scale models and prototypes. The beams with L/d ratios of 4.67 and 3.88 failed along initially formed inclined cracks with secondary splitting along the tensile reinforcement. The prototypes and 1/4-scale replica models with the L/d ratios of 2.80 and 2.00 showed multiple cracking, and in some instances failure occurred along a suddenly developed crack. The 1/2- and 1/4-scale dissimilar-strength models with L/d ratios of 2.80 and 2.00 showed no multiple cracking, and failure of these beams occurred along an initially formed crack. This difference in crack formation and propagation in the deeper dissimilar-strength models (L/d = 2.88 and 2.00,

Figure 4) accounts for the lower load-carrying capability since arch action was not fully developed.

The tensile steel strains at midspan, shown as dimensionless plots in Figures 11 and 12, indicate good correlation for the 1/4-scale replica model and prototype. The strains for the 1/4-scale dissimilar models are higher, probably due to the distortions in strength scaling. Load-deflection curves at midspan are shown in Figures 13 through 16. Although no abrupt changes in deflection rate were evident at the cracking load, the load-deflection curves and tensile and compressive steel strains were used to determine cracking loads for the Series A and B beams. Cracking loads in the deeper beams corresponded to a point at which the slope of the load-steel strain curve becomes constant.

The crack pattern of a Series A prototype beam, tested at a dynamic load approximately 50 percent over the static capacity, is compared with the crack pattern from the companion static test in Figure 17. The dynamic failure is symmetric, with much higher strains and displacements than occurred in the static tests. The dynamic load, displacements, and strains are shown in Figures 18 and 19. The oscillations in the load are characteristic of the loader. The tensile strains become uniform at approximately 6 msec, indicating arch formation; failure occurred at approximately 11 msec as evidenced by a rapid decrease in strain and an increased deflection rate.

Comparison of static and dynamic failures for the Series B prototype beams is shown in Figure 20. The dynamic test was conducted at a load slightly higher than the static capacity. The failures appear identical, but the strains and deflections under the dynamic load, shown in Figures 21 and 22, are higher than the strains and deflection from the companion static test. The dynamic data indicate completely formed arch action at approximately 8 msec, after which the beam maintains constant load with no increasing strains or deflection until failure occurs at approximately 42 msec. The time for which the load was maintained before failure occurred indicates that a lower load may not have caused failure. The increase in the dynamic load-carrying capacity may be explained by the increase in concrete strengths due to strain rate sensitivity. If a 15 to 20 percent increase in concrete strength is assumed, the static and dynamic ultimate normalized shear strengths are almost identical for the Series B prototype tests.

## CONCLUSIONS

The conclusions presented are based on limited test data reported herein.

1. From test data interpretation, the inclined cracking load can be adequately predicted from 1/4-scale replica and 1/4- and 1/2-scale dissimilar-strength models.

2. The ultimate load-carrying capacity, failure pattern, and strains of the prototype beams can be adequately predicted from 1/4-scale replica models.

3. The failure pattern and ultimate load capacity of prototype beams with L/d ratios of 4.67 and 3.88 can be predicted from the 1/4- and 1/2-scale dissimilar models. For the deeper beams (L/d = 2.80 and 2.00) the ultimate capacity of the prototypes would be underpredicted from the dissimilar-strength models.

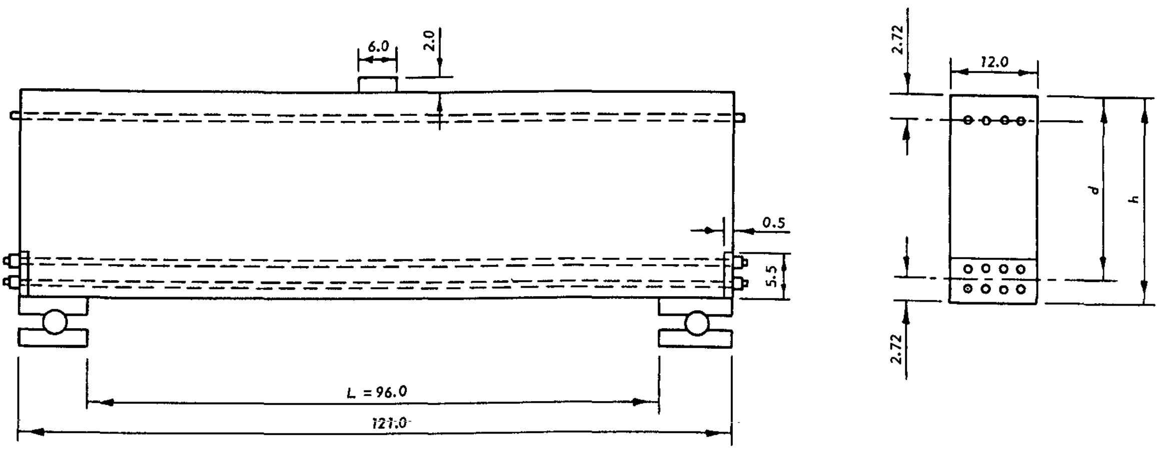
4. A comparison of static and dynamic failures and load-carrying capacities indicates that the Series A beam (L/d = 4.67), when subjected to a dynamic load approximately 50 percent higher than its static capacity, exhibited a different failure pattern. For the Series B tests (L/d = 3.88), the failures and crack patterns were very similar for static and dynamic loads with the exception that the failure crack in the dynamic test was inclined at a steeper angle to a horizontal plane. If an increase in concrete strength of 15 to 20 percent is assumed for the beam tested dynamically, the normalized ultimate shear strengths for static and dynamic loads are almost identical.

*TABLE 1 PROPERTIES OF STATIC TEST SPECIMENS*

| Beam | Beam Width b | Total Depth h | Clear Span L | Effective Depth d | $\frac{L}{d}$ | Concrete Compressive Strength $f'_c$ | Compressive Reinforcement | | | Tensile Reinforcement | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Number | Size | Ratio $p'$ | Number | Size | Ratio $p$ |
| | inches | inches | inches | inches | | psi | | | | | | |
| AE4S1 | 3 | 5.82 | 24 | 5.14 | 4.67 | 1,050 | 2 | No. 2 | 0.0065 | 4 | No. 2 | 0.0130 |
| AE4S2 | 3 | 5.82 | 24 | 5.14 | 4.67 | 1,050 | 2 | No. 2 | 0.0065 | 4 | No. 2 | 0.0130 |
| AM4S1 | 3 | 5.82 | 24 | 5.14 | 4.67 | 3,980 | 4 | No. 2 | 0.0130 | 8 | No. 2 | 0.0260 |
| AE2S1 | 6 | 11.64 | 48 | 10.28 | 4.67 | 1,983 | 3 | No. 4 | 0.0097 | 6 | No. 4 | 0.0194 |
| AP1S2 | 12 | 23.28 | 96 | 20.56 | 4.67 | 3,577 | 4 | No. 8 | 0.0130 | 8 | No. 8 | 0.0260 |
| BE4S1 | 3 | 6.87 | 24 | 6.19 | 3.88 | 1,097 | 2 | No. 2 | 0.0053 | 4 | No. 2 | 0.0106 |
| BM4S1 | 3 | 6.87 | 24 | 6.19 | 3.88 | 3,980 | 4 | No. 2 | 0.0106 | 8 | No. 2 | 0.0212 |
| BE2S1 | 6 | 13.74 | 48 | 12.38 | 3.88 | 1,970 | 3 | No. 4 | 0.0081 | 6 | No. 4 | 0.0162 |
| BP1S1 | 12 | 27.48 | 96 | 24.76 | 3.88 | 4,340 | 4 | No. 8 | 0.0106 | 8 | No. 8 | 0.0212 |
| CE4S1 | 3 | 9.25 | 24 | 8.57 | 2.80 | 1,097 | 2 | No. 2 | 0.0039 | 4 | No. 2 | 0.0078 |
| CM4S1 | 3 | 9.25 | 24 | 8.57 | 2.80 | 3,980 | 4 | No. 2 | 0.0077 | 8 | No. 2 | 0.0154 |
| CE2S1 | 6 | 18.50 | 48 | 17.14 | 2.80 | 2,030 | 3 | No. 4 | 0.0058 | 6 | No. 4 | 0.0117 |
| CP1S1 | 12 | 37.00 | 96 | 34.28 | 2.80 | 3,893 | 4 | No. 8 | 0.0077 | 8 | No. 8 | 0.0154 |
| CP1S3 | 12 | 37.00 | 96 | 34.28 | 2.80 | 3,980 | 4 | No. 8 | 0.0077 | 8 | No. 8 | 0.0154 |
| DE4S2 | 3 | 12.68 | 24 | 12.00 | 2.00 | 920 | 2 | No. 2 | 0.0028 | 4 | No. 2 | 0.0056 |
| DE4S3 | 3 | 12.68 | 24 | 12.00 | 2.00 | 920 | 2 | No. 2 | 0.0028 | 4 | No. 2 | 0.0056 |
| DM4S3 | 3 | 12.68 | 24 | 12.00 | 2.00 | 3,980 | 4 | No. 2 | 0.0056 | 8 | No. 2 | 0.0112 |
| DE2S1 | 6 | 25.36 | 48 | 24.00 | 2.00 | 2,020 | 3 | No. 4 | 0.0042 | 6 | No. 4 | 0.0083 |
| DP1S1 | 12 | 50.72 | 96 | 48.00 | 2.00 | 4,343 | 4 | No. 8 | 0.0055 | 8 | No. 8 | 0.0110 |
| DP1S2 | 12 | 50.72 | 96 | 48.00 | 2.00 | 3,946 | 4 | No. 8 | 0.0055 | 8 | No. 8 | 0.0110 |

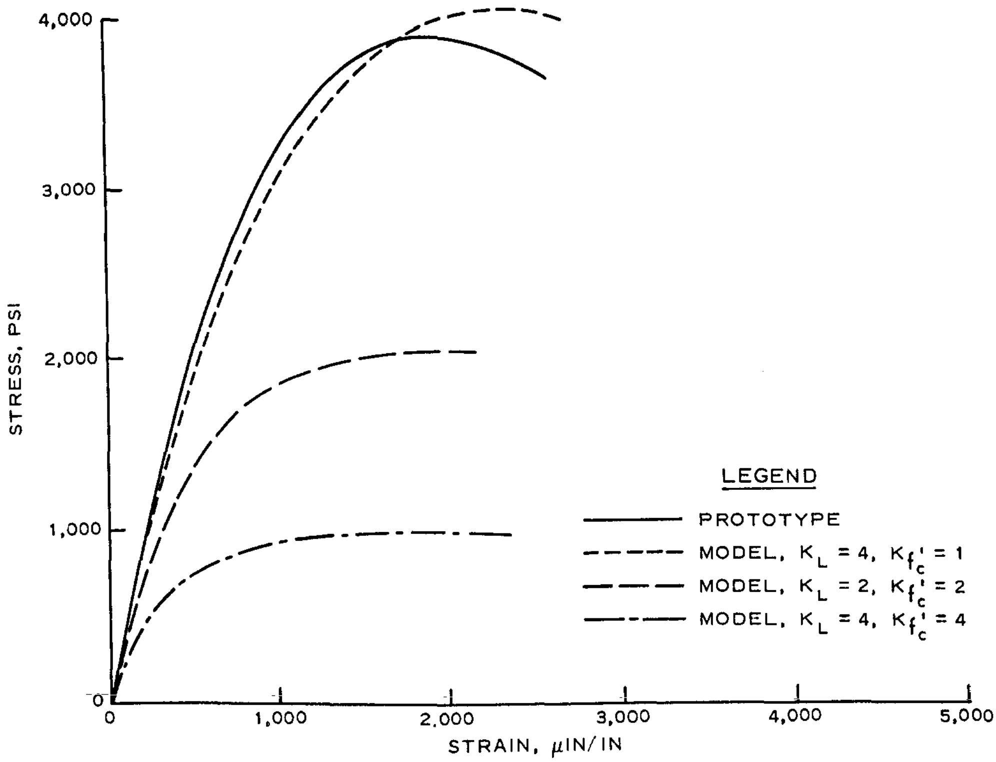
TABLE 2 EXPERIMENTAL CRACKING AND ULTIMATE SHEAR

| Beam | Concrete Compressive Strength $f'_c$ | Cracking Shear $V_c$ | Ultimate Shear $V_u$ | Cracking Shear Parameter $V_c/bd\sqrt{f'_c}$ | Ultimate Shear Parameter $V_u/bd\sqrt{f'_c}$ |
|---|---|---|---|---|---|
| | psi | kips | kips | | |
| AE4S1 | 1,050 | 1.50 | 1.96 | 3.00 | 3.92 |
| AE4S2 | 1,050 | 1.85 | 2.13 | 3.70 | 4.26 |
| AM4S1 | 3,980 | 3.23 | 4.30 | 3.32 | 4.42 |
| AE2S1 | 1,983 | 9.50 | 11.10 | 3.46 | 4.04 |
| AP1S2 | 3,577 | 37.00 | 53.50 | 2.51 | 3.63 |
| BE4S1 | 1,097 | 1.90 | 2.27 | 3.09 | 3.65 |
| BM4S1 | 3,980 | 4.35 | 5.38 | 3.70 | 4.59 |
| BE2S1 | 1,970 | -- | 9.50 | -- | 2.88 |
| BP1S1 | 4.340 | 57.00 | 64.00 | 2.94 | 3.32 |
| CE4S1 | 1,097 | 2.03 | 3.60 | 2.38 | 4.23 |
| CM4S1 | 3,980 | 5.80 | 11.71 | 3.57 | 7.22 |
| CE2S1 | 2,030 | 13.75 | 17.15 | 3.01 | 3.70 |
| CP1S1 | 3,893 | 80.00 | 207.00 | 3.12 | 8.06 |
| CP1S3 | 3,980 | 65.00 | 162.00 | 2.38 | 5.95 |
| DE4S2 | 920 | 3.25 | 5.55 | 2.98 | 5.08 |
| DE4S3 | 920 | -- | 5.21 | -- | 4.77 |
| DM4S3 | 3,980 | 7.10 | 14.61 | 3.13 | 6.54 |
| DE2S1 | 2,020 | -- | 33.10 | -- | 5.06 |
| DP1S1 | 4,343 | 100.00 | 248.00 | 2.63 | 6.53 |
| DP1S2 | 3,946 | 102.50 | 257.50 | 2.83 | 7.12 |

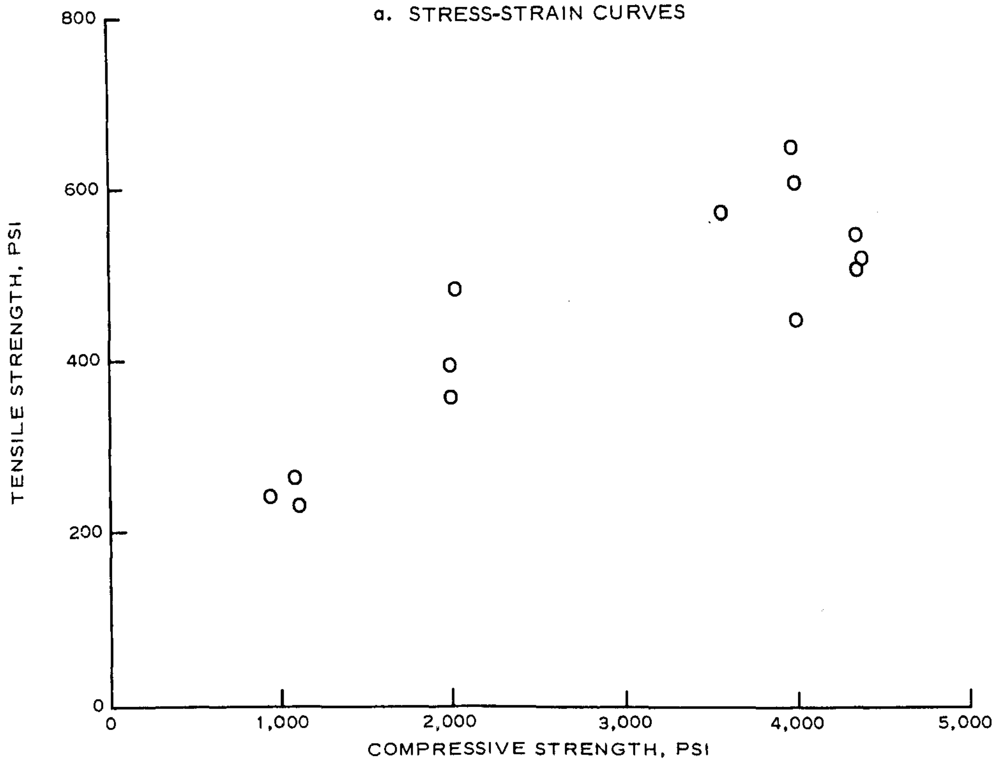



| SERIES | A | B | C | D |
|---|---|---|---|---|
| d | 20.56 | 24.76 | 34.28 | 48.00 |
| h | 23.28 | 27.48 | 37.00 | 50.72 |
| L/d | 4.67 | 3.38 | 2.80 | 2.00 |

Figure 1 Dimensions (in inches) for simply supported prototype deep beams.




a. STRESS-STRAIN CURVES




b. TENSILE STRENGTH-COMPRESSIVE STRENGTH RELATION

Figure 2 Concrete properties.

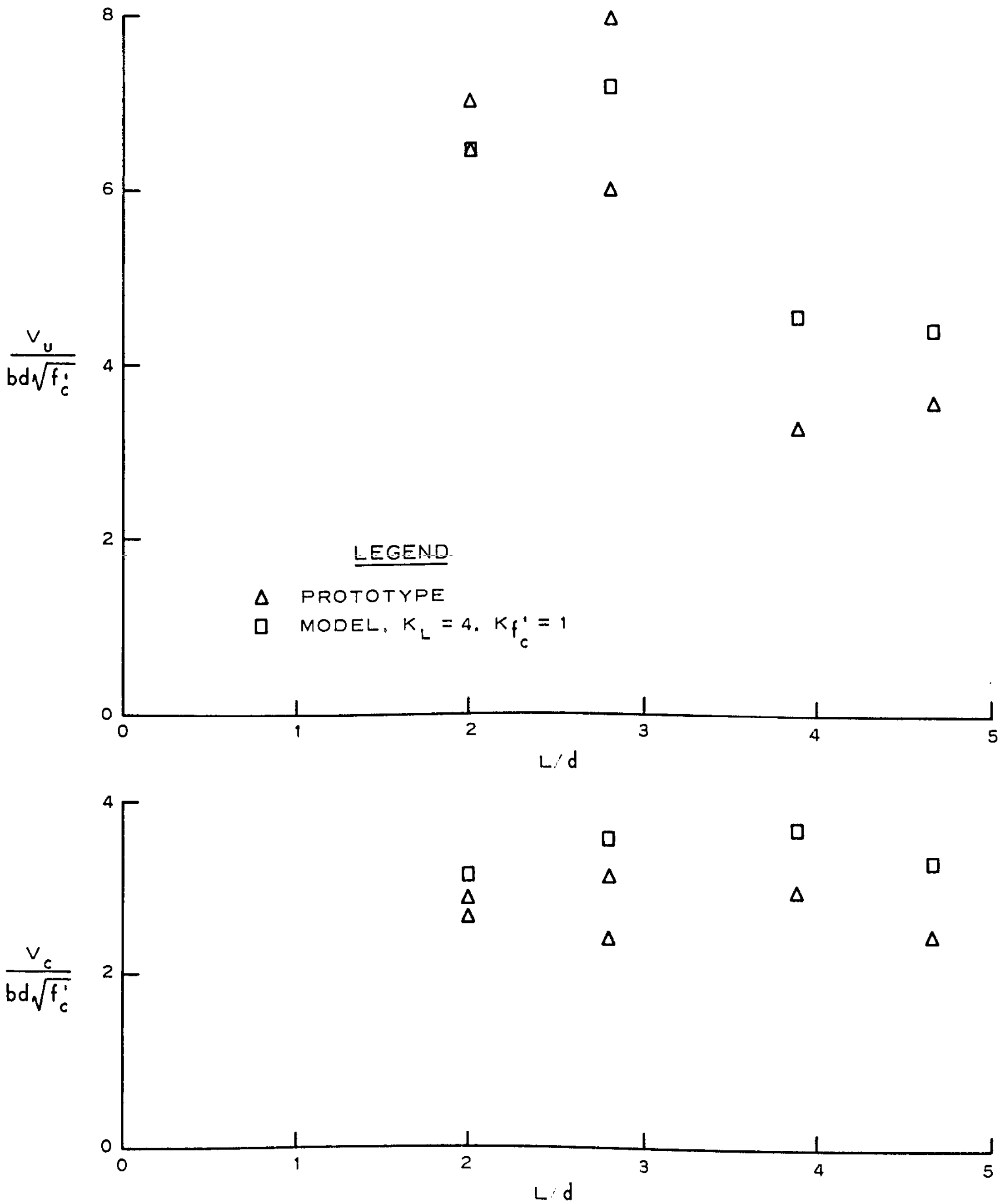



Figure 3 Inclined cracking and ultimate shear of replica models and prototypes.

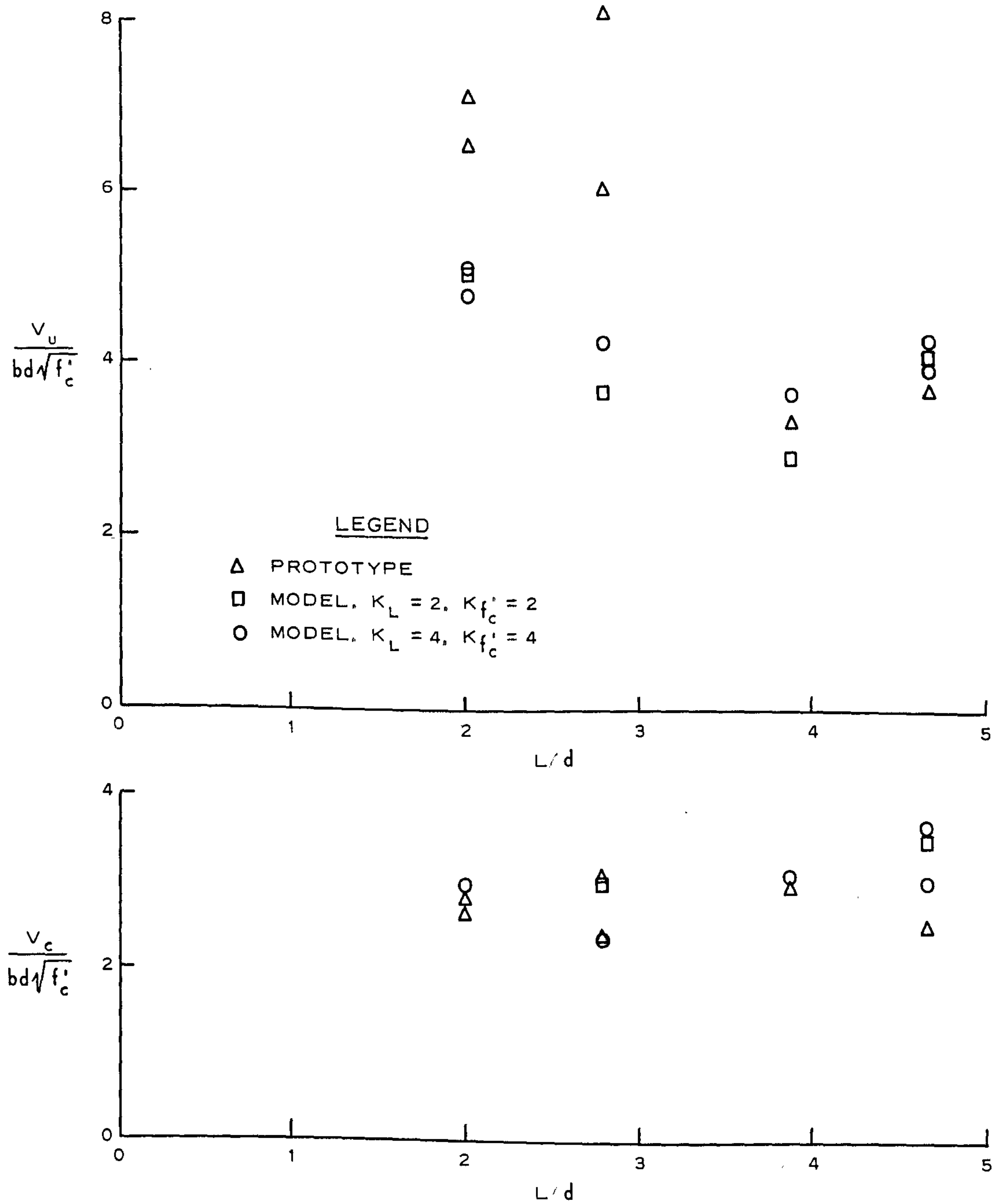



Figure 4 Inclined cracking and ultimate shear of dissimilar-strength models and prototypes.

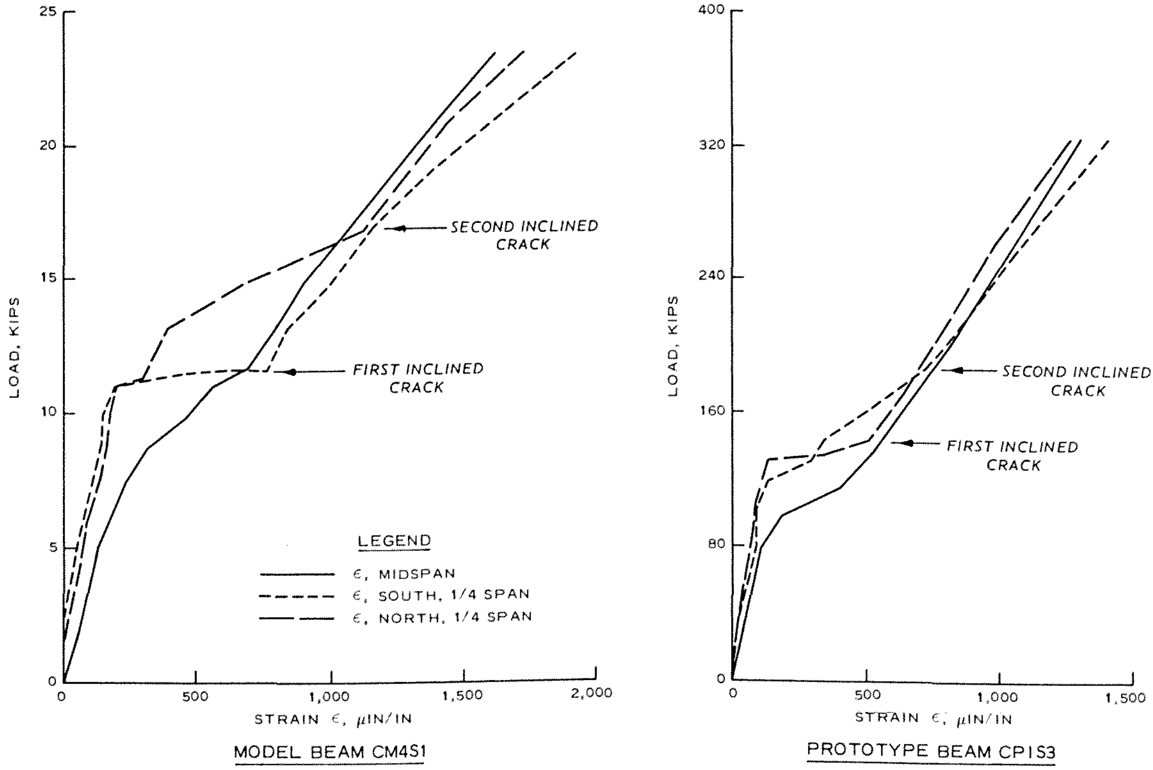



Figure 5 Inclined cracking loads determined from tensile steel strains, Series C replica model and prototype.

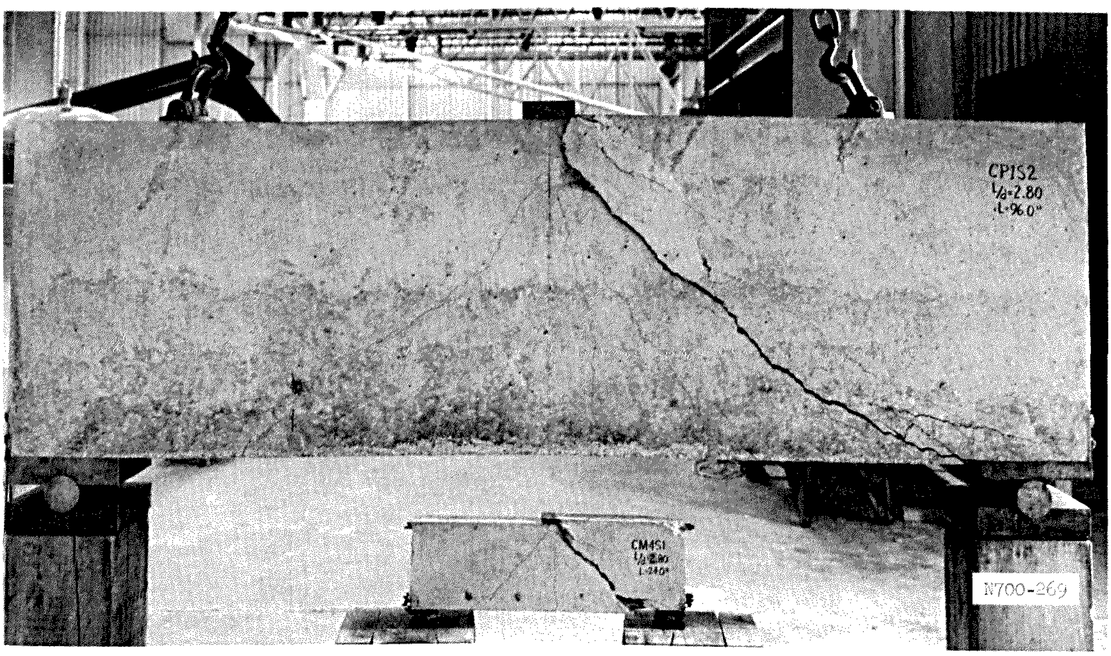
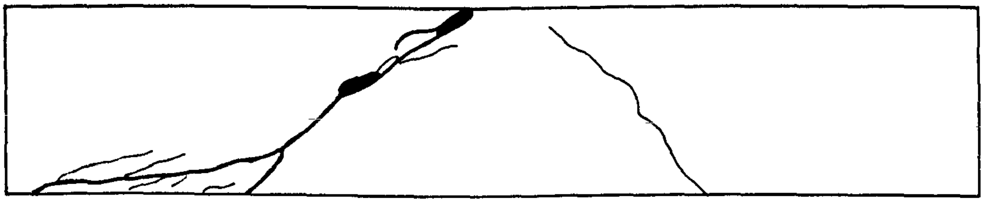
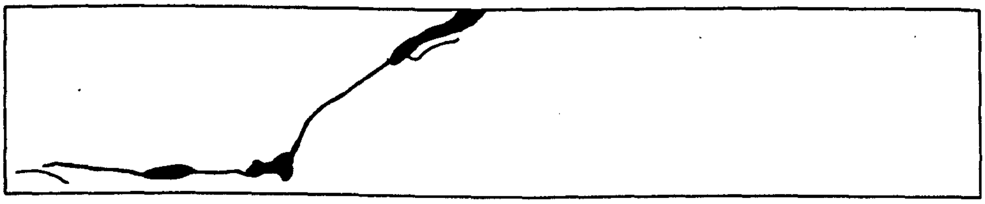



Figure 6 Crack patterns of 1/4-scale replica model and prototype.

a. Beam AP1S2, prototype.

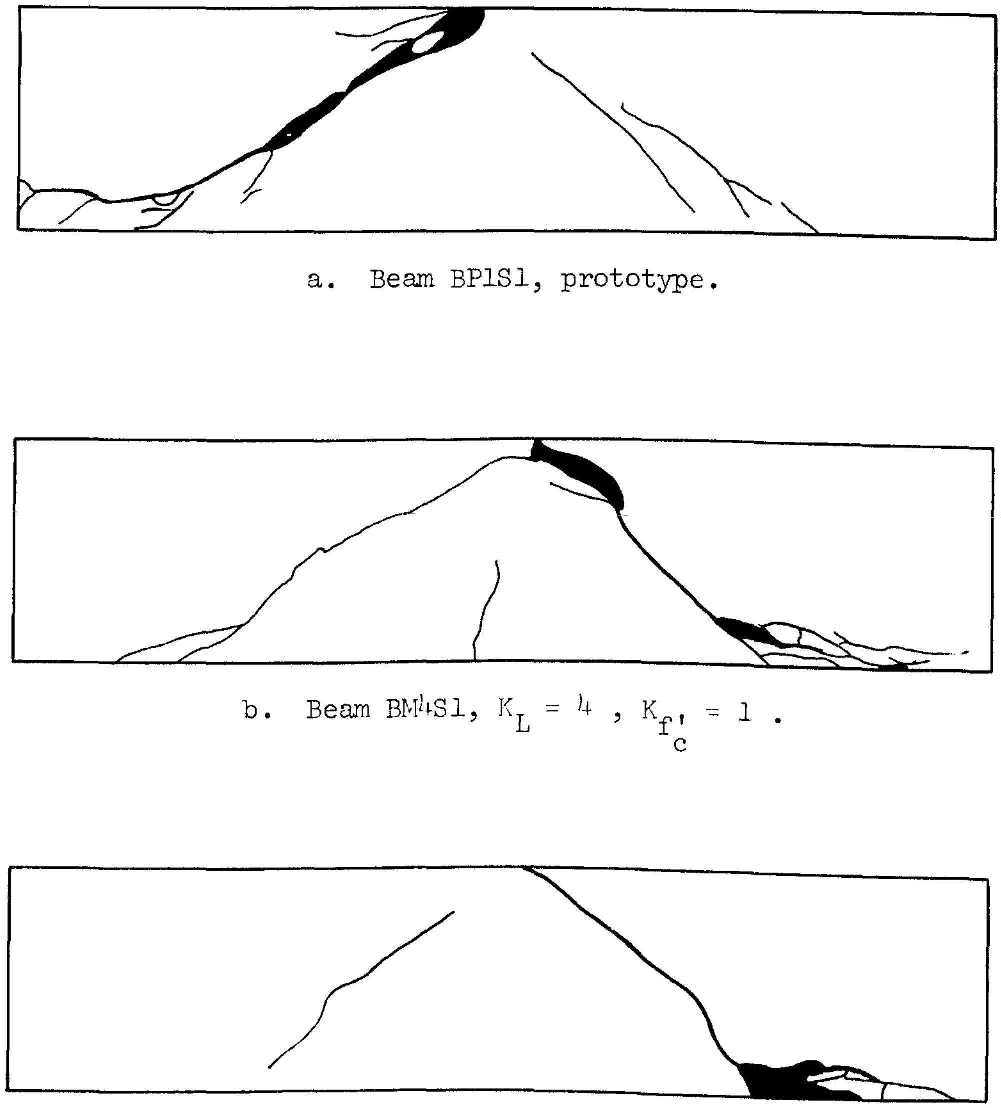


b. Beam AM4S1, $K_L = 4$ , $K_{f'_c} = 1$ .



c. Beam AE4S1, $K_L = 4$ , $K_{f'_c} = 4$ .

Figure 7 Crack patterns of Series A 1/4-scale models and prototype.

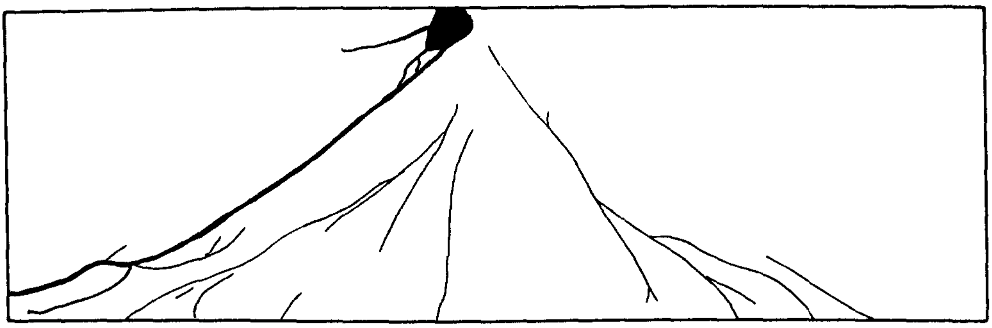


a. Beam BP1S1, prototype.

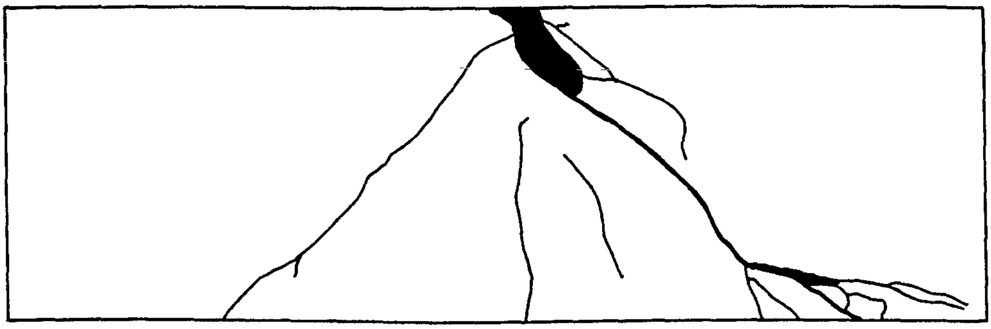
b. Beam BM4S1, $K_L = 4$ , $K_{f'_c} = 1$ .

c. Beam BE4S1, $K_L = 4$ , $K_{f'_c} = 4$ .

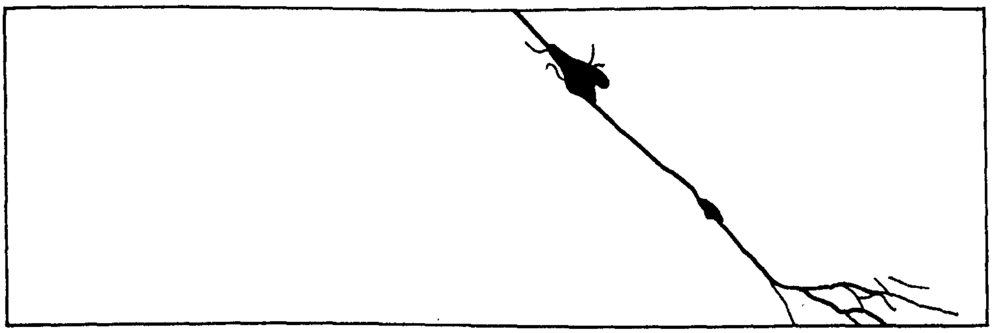
Figure 8 Crack patterns of Series B 1/4-scale models and prototype.



a. Beam CP1S3, prototype.



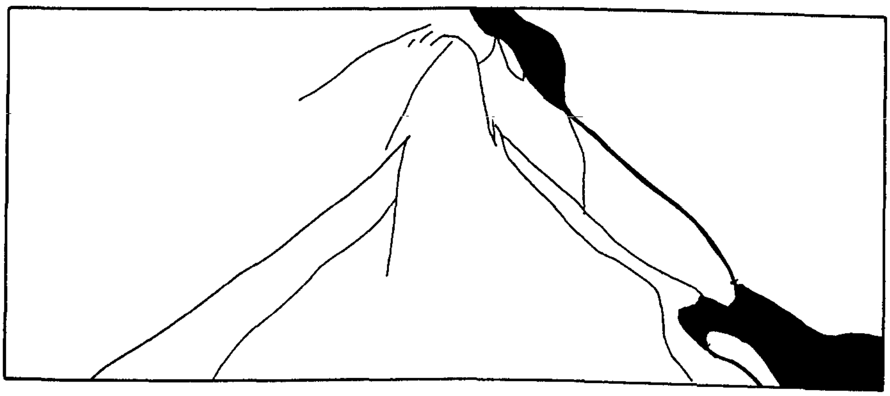
b. Beam CM4S1, $K_L = 4$ , $K_{f'_c} = 1$ .



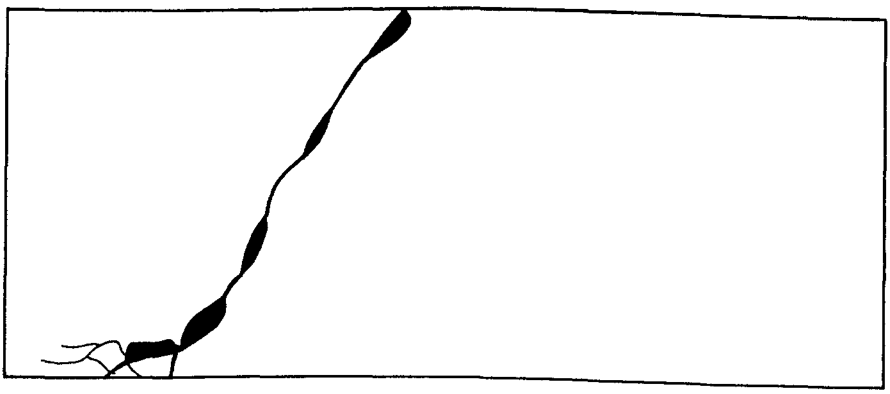
c. Beam CE4S1, $K_L = 4$ , $K_{f'_c} = 4$ .

Figure 9 Crack patterns of Series C 1/4-scale models and prototype.

a. Beam DP1S2, prototype.



b. Beam DM4S3, $K_L = 4$ , $K_{f'_c} = 1$ .



c. Beam DE4S3, $K_L = 4$ , $K_{f'_c} = 4$ .

Figure 10 Crack patterns of Series D 1/4-scale models and prototype.

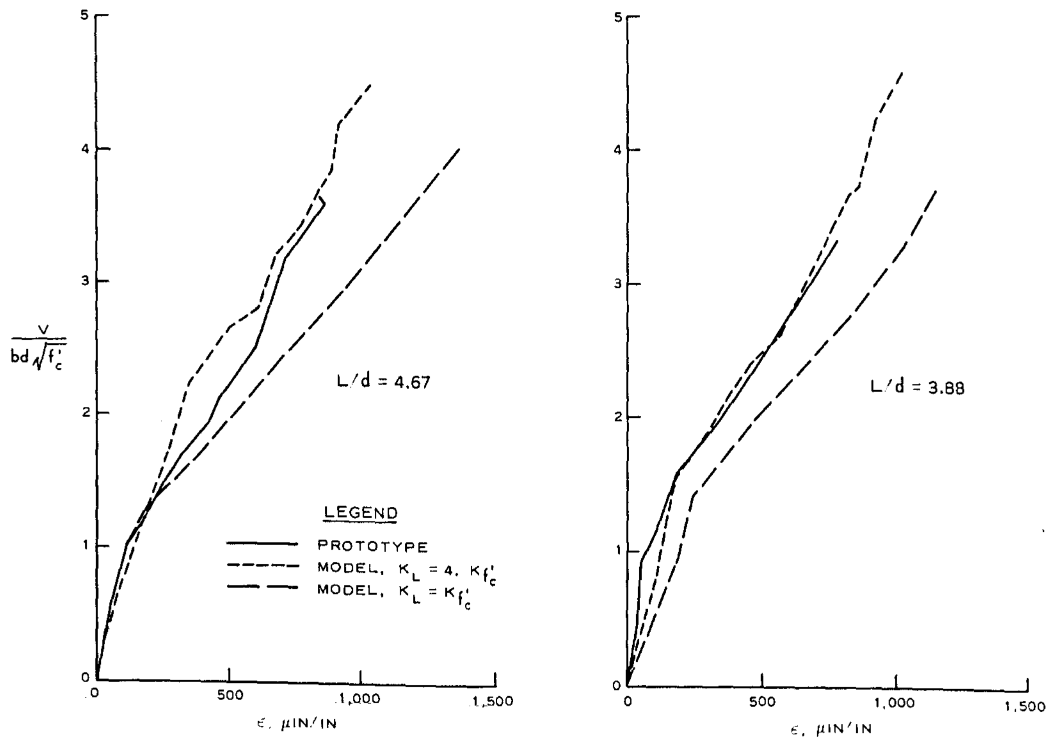



Figure 11 Normalized shear-tensile steel strains at midspan for Series A and B 1/4-scale models and prototypes.

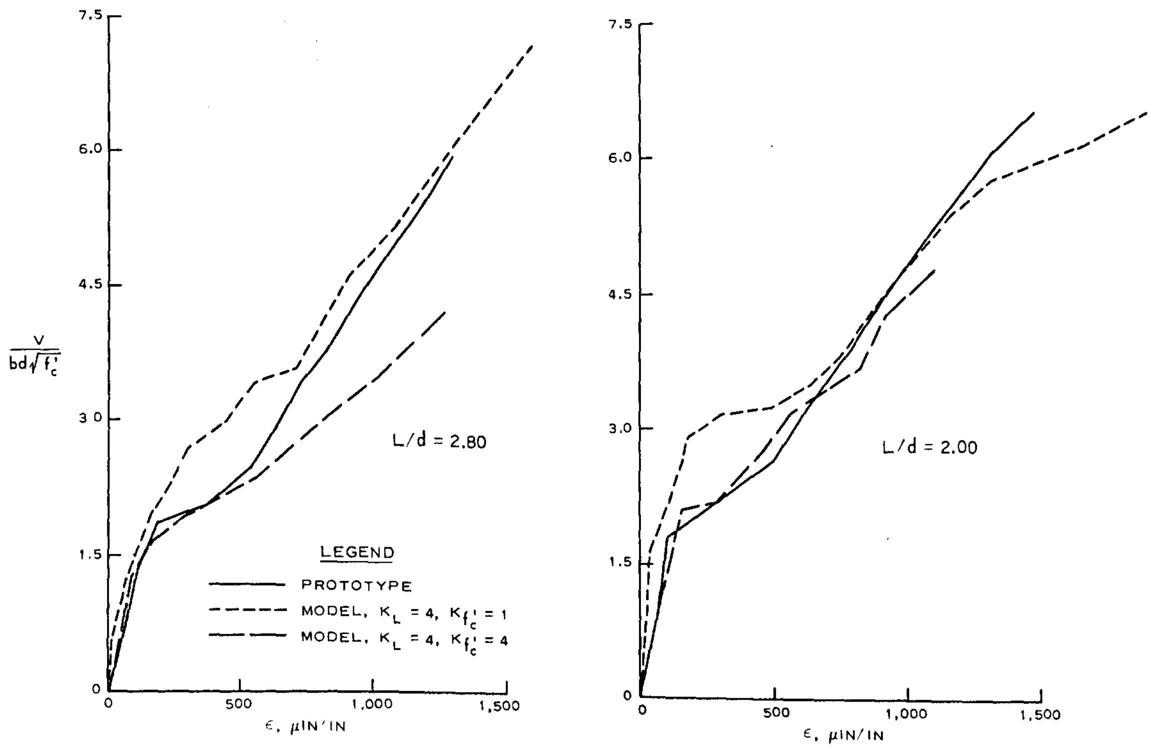



Figure 12 Normalized shear-tensile steel strains at midspan for Series C and D 1/4-scale models and prototypes.

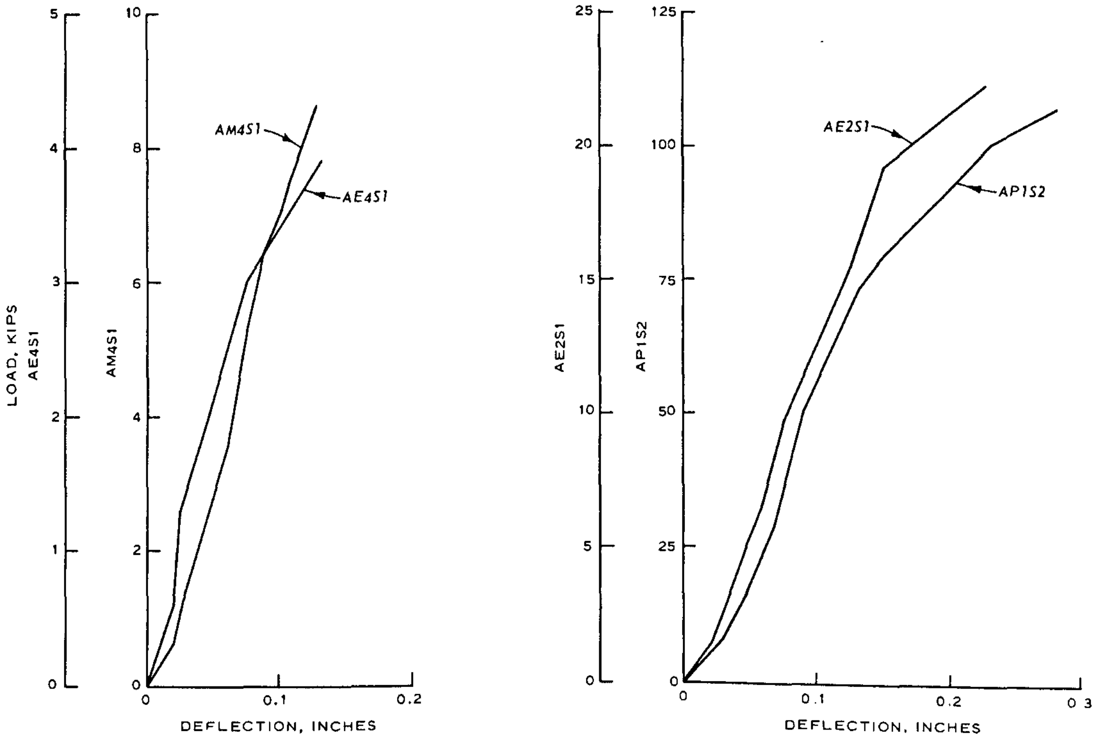



Figure 13 Load-deflection at midspan for Series A 1/4-scale models and prototype.

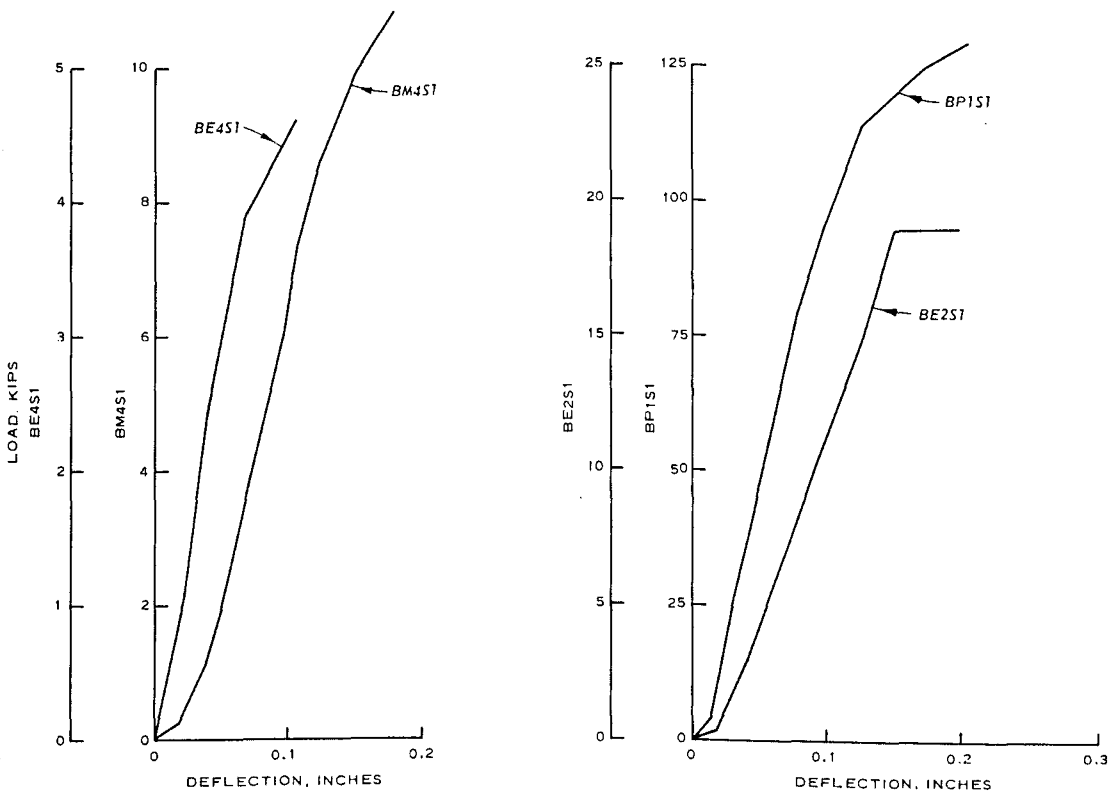



Figure 14 Load-deflection at midspan for Series B 1/4-scale models and prototype.

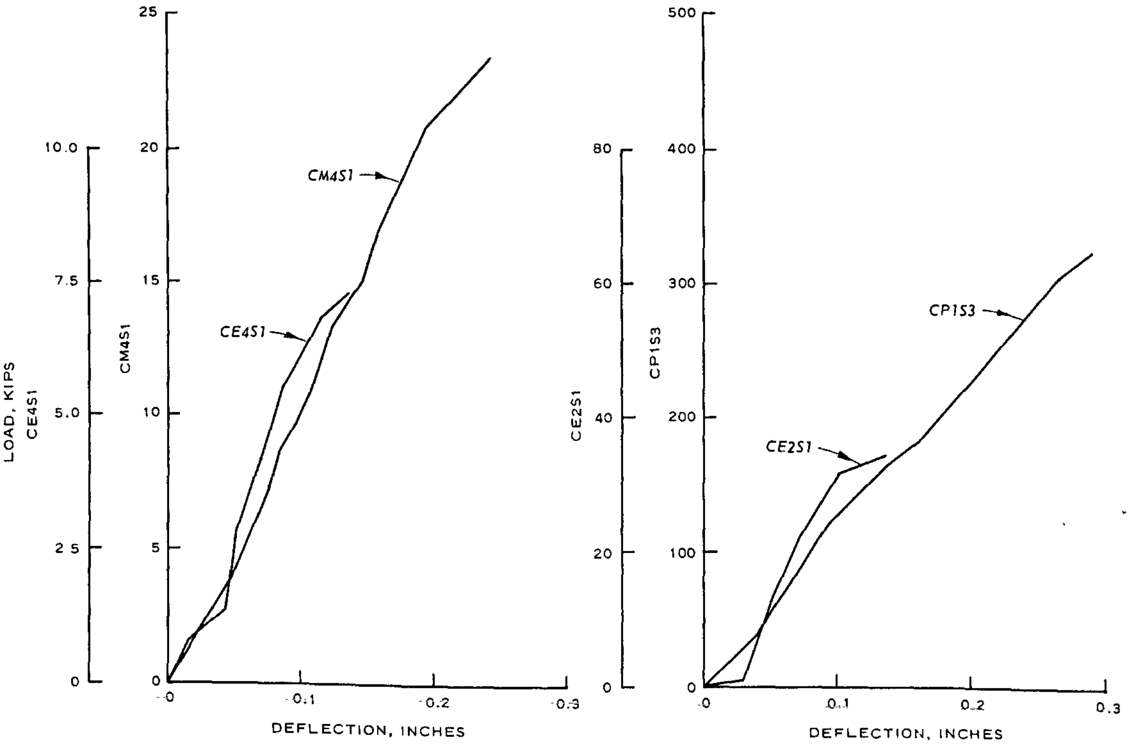



Figure 15 Load-deflection at midspan for Series C 1/4-scale models and prototype.

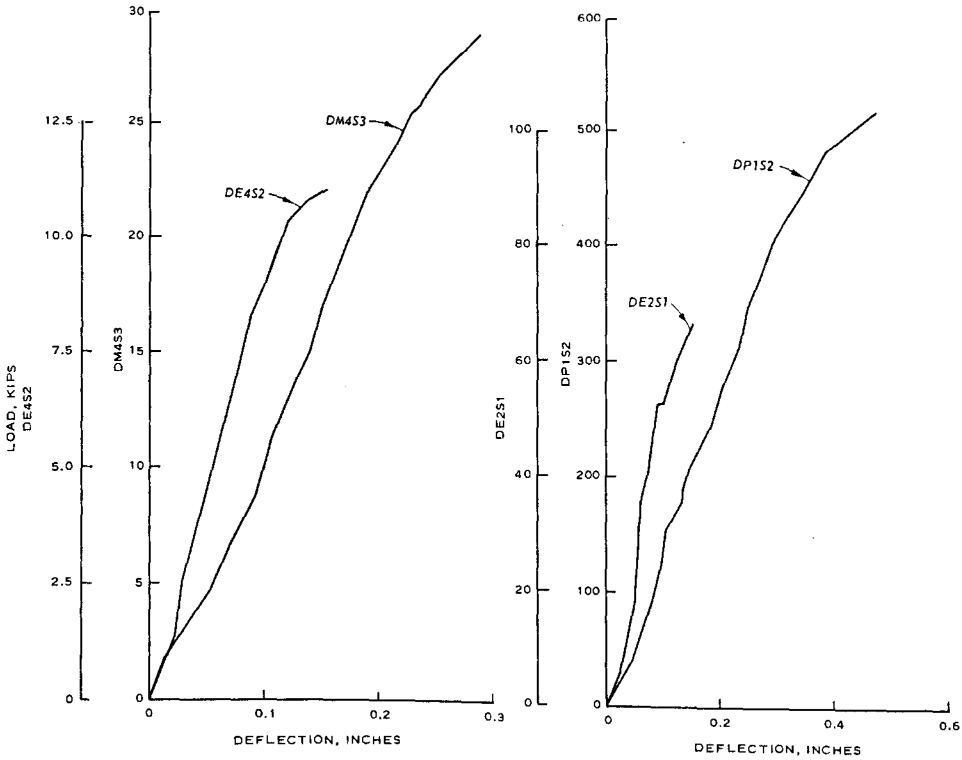
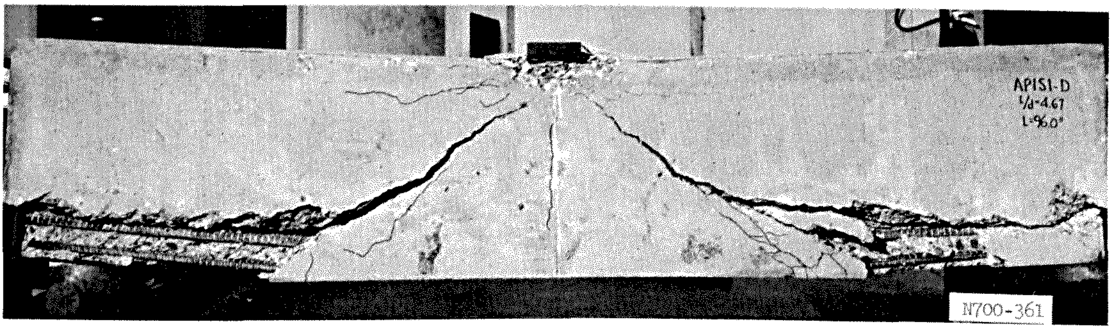
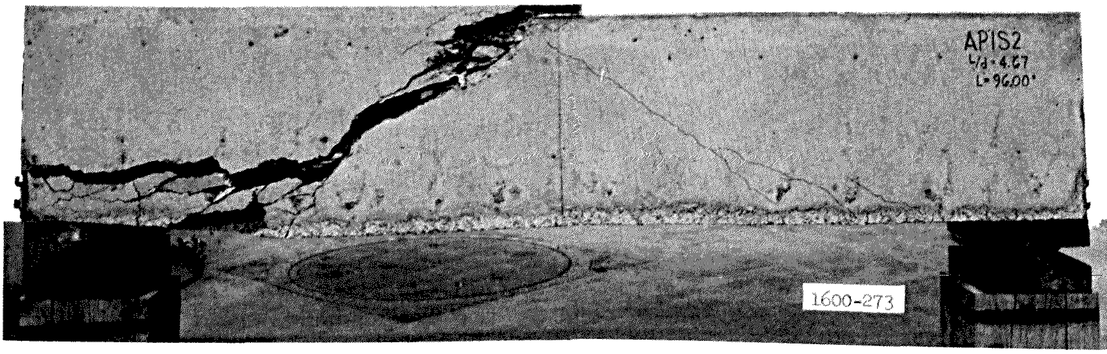



Figure 16 Load-deflection at midspan for Series D 1/4-scale models and prototype.




a. Dynamic test, $P_u$ = 155 kips , $f'_c$ = 3,957 psi.




b. Static test, $P_u$ = 107 kips , $f'_c$ = 3,577 psi.

Figure 17 Comparison of crack patterns of Series A prototypes after static and dynamic tests.

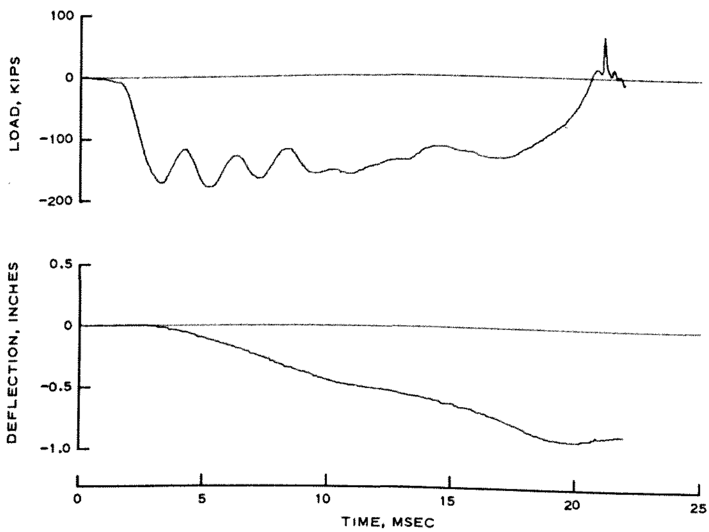



Figure 18 Dynamic load and midspan deflection for Series A prototype beam.

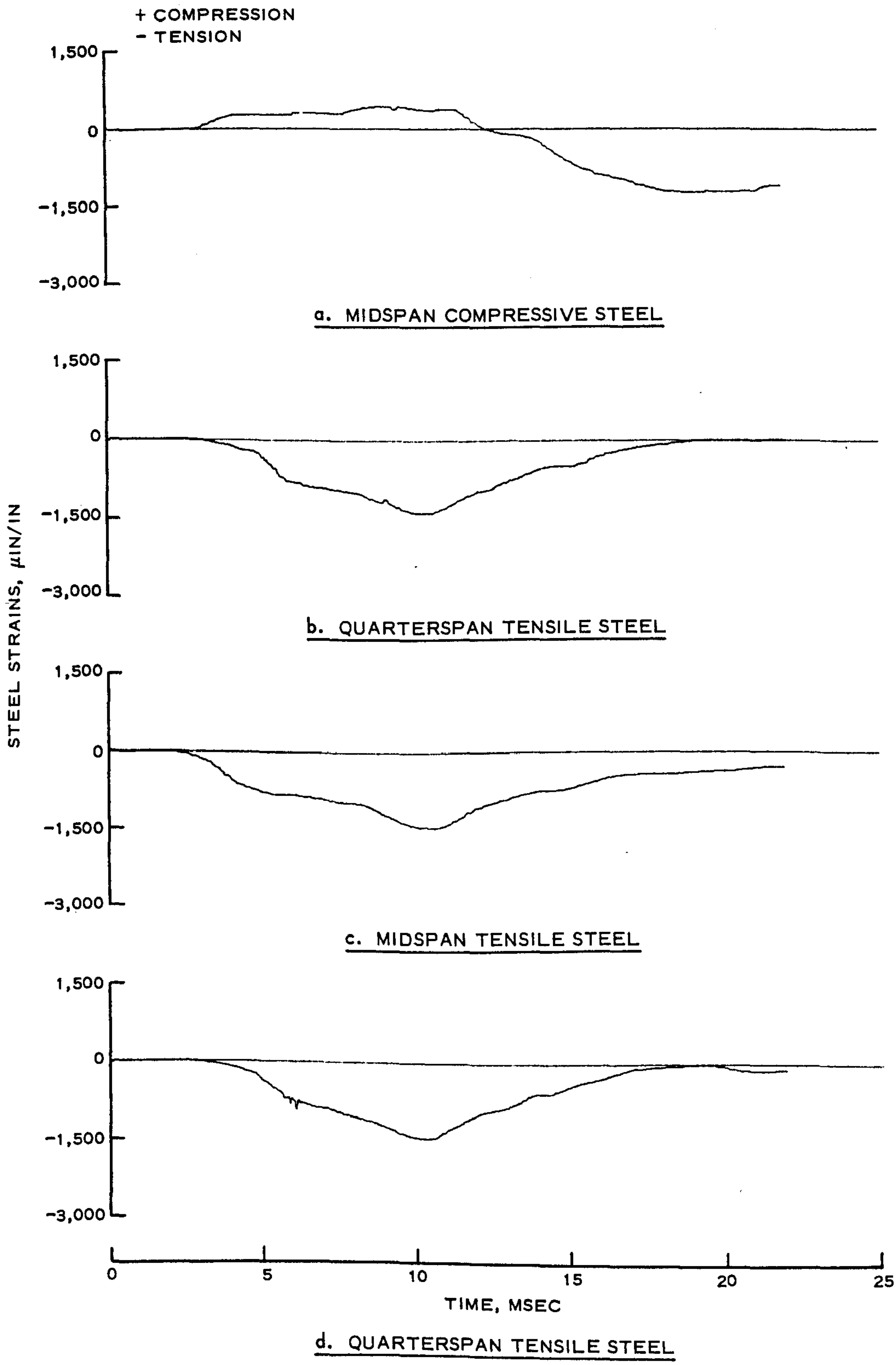
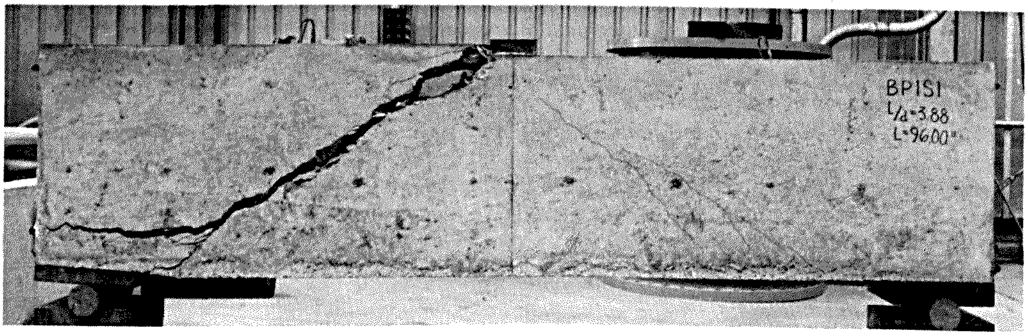
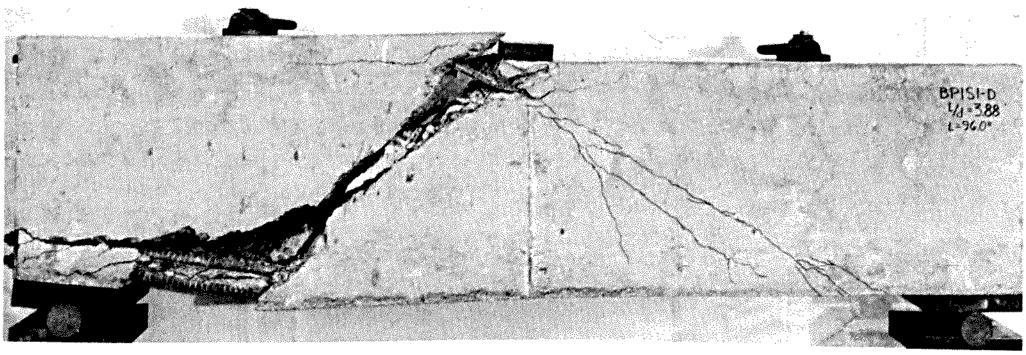



Figure 19 Dynamic steel strains for Series A prototype beam.



a. Dynamic test, $P_u$ = 135 kips, $f'_c$ = 3,730 psi.



b. Static test, $P_u$ = 128 kips, $f'_c$ = 4,340 psi.

Figure 20 Comparison of crack patterns of Series B prototypes after static and dynamic tests.

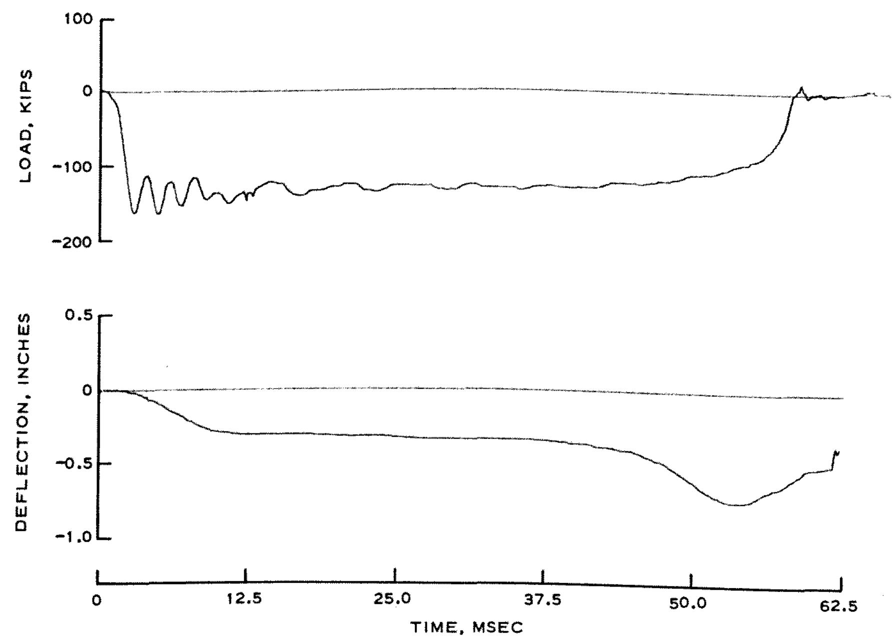



Figure 21 Dynamic load and midspan deflection for Series B prototype beam.

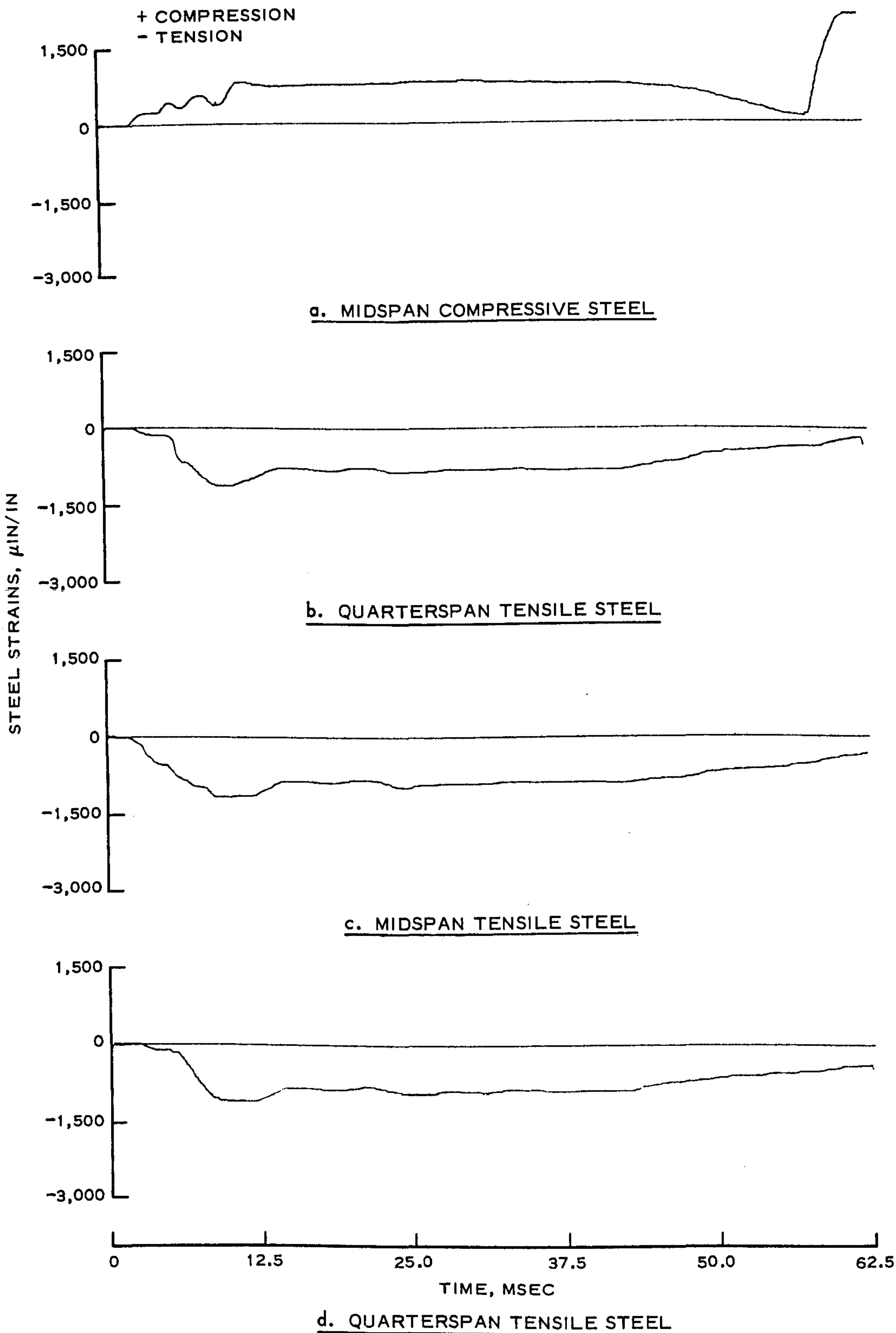



Figure 22 Dynamic steel strains for Series B prototype beam.

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13. ABSTRACT

Tests were performed on deep beams under a midspan load to determine the scaling of cracking and ultimate load-carrying capacities of beams failing in shear. Two types of scaling procedures were used, one in which only the geometries are scaled (replica or mach models), the other in which both geometry and material properties are scaled (dissimilar-strength or environmental models). The results of twenty simply supported beams tested statically with span-to-depth ratios of 4.67, 3.88, 2.80, and 2.00 and comprising 1/4- and 1/2-scale models and laboratory prototypes are presented. Two prototype beams with L/d ratios of 4.67 and 3.88 were tested dynamically to provide some correlation between statically and dynamically loaded beams. Test results indicate that cracking loads can be adequately predicted from both replica and dissimilar-strength models and ultimate loads can be predicted from replica models for all span-to-depth ratios tested. When transition from beam to arch action occurs, the dissimilar-strength models underpredict the ultimate load-carrying capacity of the prototypes.

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| 14. KEY WORDS | LINK A | | LINK B | | LINK C | |
|---|---|---|---|---|---|---|
| | ROLE | WT | ROLE | WT | ROLE | WT |
| Beams | | | | | | |
| Prototypes | | | | | | |
| Reinforced concrete | | | | | | |
| Scaling | | | | | | |
| Similitude | | | | | | |



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